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ADVANCED WASTE TREATMENT AND WATER REUSE SYMPOSIUM HELD AT CLEVELAND ENGINEERING AND SCIENTIFIC CENTER, CLEVELAND, OHIO, ON MARCH 30-31, 1971. VOLUME II

James E. Laughlin

Cleveland Engineering and Scientific Center
Cleveland, Ohio

March 1971

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CLEAN WATER

VOLUME II ADVANCED WASTE TREATMENT AND WATER REUSE SYMPOSIUM

Cleveland Engineering and Scientific Center
Cleveland, Ohio
MARCH 30-31, 1971

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MODIFICATION OF A TRICKLING FILTER PLANT TO ALLOW CHEMICAL PRECIPITATION

*James E. Laughlin, P. E.

This paper is intended to give the reader information on how a trickling filter waste treatment plant was modified to permit chemical precipitation of phosphorous and other substances. Also, some very preliminary performance data are included from the operational phase of this Advanced Waste Treatment Project.

One performance objective in this project is reduction of phosphorous (as P) to a level of one mg/l or less. Reduction of BOD and suspended solids to 10 or 15 mg/l are further goals of the project. If this plant can perform that well, consistently and economically, a contribution may be made toward enhancing performance of thousands of other trickling filter plants in use today.

Both the modifications described and subsequent performance studies are part of a 28-month Advanced Waste Treatment Project. Physical modifications are complete. Operational investigations are underway and will continue through 1971. By early 1972 it should be possible to assess whether the modifications were well conceived, and whether the process can be optimized and made part of the plant operating procedure. The appendices to this paper

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*Partner, Shimek-Roming-Jacobs & Finklea, Consulting Engineers, Dallas, Texas

This paper describes part of an Advanced Waste Treatment Project now underway at the City of Richardson, Texas. Mr. Robert E. Derrington, Water Superintendent, is Project Director. Mr. Laughlin is Associate Director. The project is 75% funded by the Environmental Protection Agency.

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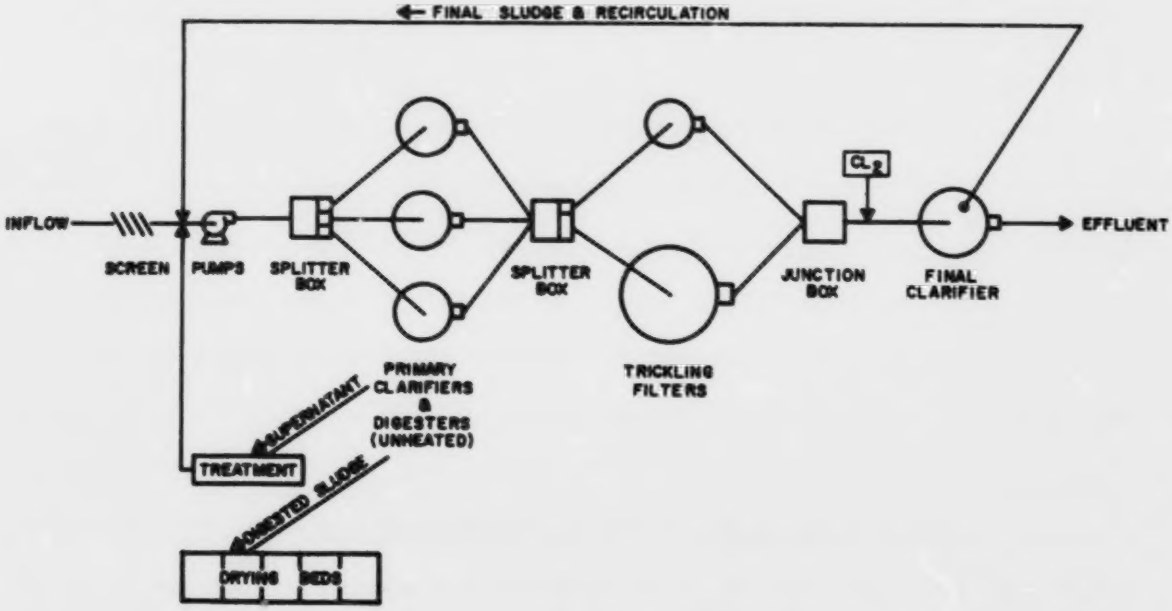
include the project schedule, and background on generation and processing of data.

EXISTING TREATMENT FACILITIES

In this project, chemical treatment is intended as an adjunct to the typical physical and biological treatment already provided. Furthermore, it was intended that required modifications be as simple as possible and that existing facilities be fully utilized. The plant's existing facilities can be shown in a schematic sketch. Design capacity is 1.6 MGD.

CITY OF RICHARDSON, TEXAS
WASTEWATER TREATMENT PLANT
1969


FINAL SLUDGE & RECIRCULATION
INFLOW
SCREEN
PUMPS
SPLITTER BOX
SPLITTER BOX
CL_2
JUNCTION BOX
FINAL CLARIFIER
EFFLUENT
PRIMARY CLARIFIERS & DIGESTERS (UNHEATED)
TRICKLING FILTERS
SUPERNATANT
TREATMENT
DIGESTED SLUDGE
DRYING BEDS

A mechanical bar screen precedes a wet well serving four raw sewage pumps which lift the water into a flow splitter box. Proportional weirs there divide flow between three clarifier-digesters. Primary effluent is combined in a splitter box, then divided and sent to two standard rate rock filters. Filter effluent is combined and carried to the final clarifier. Chlorination and settling are simultaneous in that clarifier. A mixture of final sludge and recirculation are returned to the head of the plant; the amount of recirculation is regulated by a level control system in the raw sewage wet well.

Sludge is digested in the lower compartment of each primary clarifier-digester. No heat is provided (gas is wasted through a burner) and mixing consists of gentle stirring by a 3 rph mechanism revolving on the same shaft as the clarifier rakes above.

Sludge is dried on sand beds. Filtrate collected in the underdrains goes back to the head of the plant. Digester supernatant is drawn and batch treated before return to the head of the plant. Three 500 gallon fill-and-draw tanks receive raw supernatant; alum is added, 250 mg/l, then 20 minutes of air agitation yield a finished liquor which separates into sludge (which goes onto drying beds) and treated supernatant with strength comparable to raw sewage.

Detailed plant data can be condensed into the table which follows.

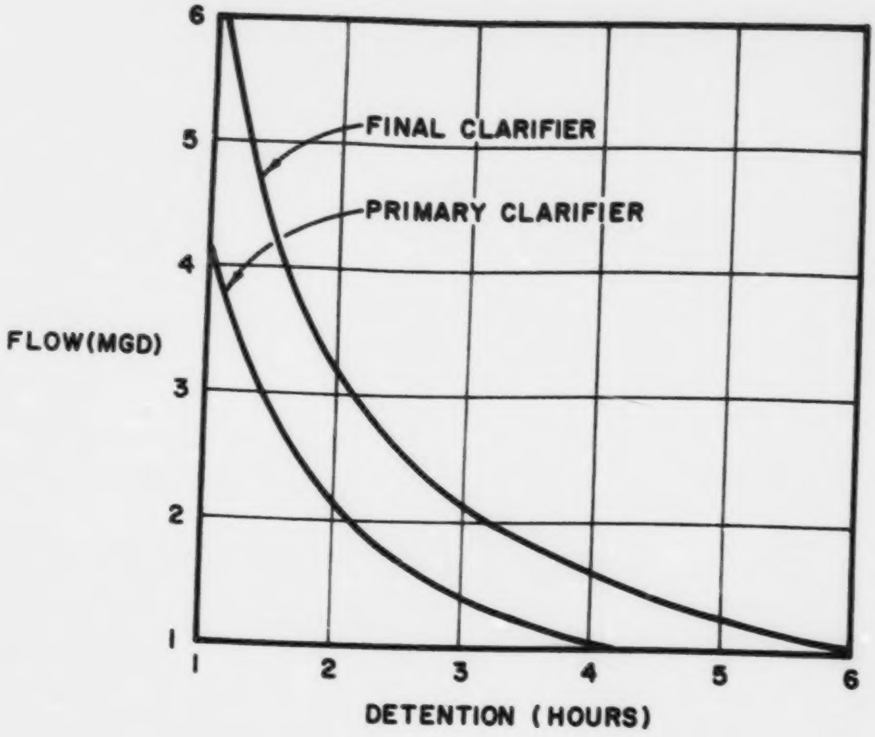
DATA ON TREATMENT UNITS

	Diam (Ft)	Depth (Ft)	Circum (Ft)	Area (Sq Ft)	Volume (Cu Ft)	Volume (Gal)
Primary Clarifier No. 1	40	8	126	1257	10,054	75,200
2	40	10	126	1257	12,570	94,000
3	40	10	126	1257	12,570	94,000
Primary Clarifiers Combined	--	--	378	3771	35,194	263,200
Final Clarifier	70	6	220	3848	23,088	173,000
Filter No. 1	84	6.5	--	5542(1)	36,000	--
2	120	6.5	--	11310(1)	73,500	--
Filters Combined	--	--	--	16852(1)	109,500	--
Digester No. 1	40	14.3(2)	--	1257	13,000	135,000
2	40	14.3(2)	--	1257	13,000	135,000
3	40	14.3(2)	--	1257	13,000	135,000
Digesters Combined	--	--	--	--	39,000	404,000

Sludge Drying Beds 12,000 Square Feet

(1) Area in acres: 0.127, 0.260 and 0.387, respectively
(2) 14.3 Effective, 18.0 SWD, 15.8 Clear @Center

These physical dimensions allow estimation of clarifier detention times at different flows, an important factor in predicting lag time through the plant. Hyperbolic equations describing assumed plug flow are plotted on the following graph.



Although the plot is only an approximation of actual conditions (and this is further compounded when assuming plant detention time equals cl detention time), it has proven most helpful in predicting passage of conditions through the system.

The following table summarizes data on the load received at the existing plant, and on the performance of the system.

CHARACTERISTICS OF WASTEWATER
(mg/l unless noted)

	Typical Influent (Apr-Nov 1970)	Conventional Effluent (Apr-Aug 1970)
Flow	*1.6 MGD	
Suspended Solids	145	*15
BOD	140	*20
Phosphorous (p)	8.8	8
Phosphorous (P)	122 lb/day	
Total Kjeldahl Nitrogen (N)	21	10
Iron (Fe)	0.84	
Aluminum (Al)	*0.23	
Alkalinity (Ca CO_3)	175	
Ratio BOD/COD	0.45	

*Geometric means, all other values are arithmetic means.

Richardson's treatment plant has been attended full time for some years. Approximately 320 manhours per week are required to operate and maintain the facility. In addition, an analyst devotes 40 man hours per week in the control laboratory.

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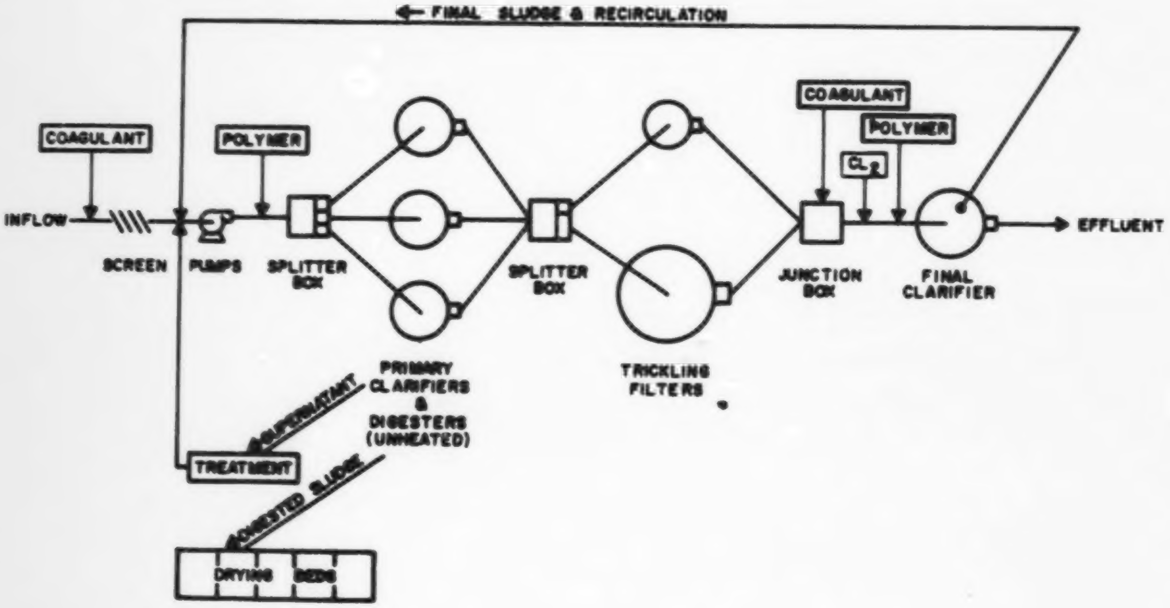
MODIFICATION FOR CHEMICAL ADDITION

Early in the project two coagulants were selected for operational trials: aluminum in the form of liquid alum, and iron in the form of liquid ferric chloride. Both are available in bulk, from commercial firms, at haul distances of about 250 miles. Both are similar enough in character to permit use of common storage and feeding hardware. Use of polymers was also predicated at this point, assuming they would be worthwhile in improving settling characteristics of solids involved.

The flow diagram of the plant could now be modified to permit addition of these chemicals at the head end and just ahead of the final clarifier.

CITY OF RICHARDSON, TEXAS
WASTEWATER TREATMENT PLANT
1970


← FINAL SLUDGE & RECIRCULATION
COAGULANT
POLYMER
COAGULANT
POLYMER
CL_2
INFLOW
SCREEN
PUMPS
SPLITTER BOX
SPLITTER BOX
JUNCTION BOX
FINAL CLARIFIER
EFFLUENT
PRIMARY CLARIFIERS & DIGESTERS (UNHEATED)
TRICKLING FILTERS
SUPERNATANT
TREATMENT
DIGESTED SLUDGE
DRYING BEDS

One 6000 gallon fiberglass stores liquid coagulant in a central location. Two chemical feed pumps are installed beside the tank, and piped to deliver to either the head or effluent ends of the plant, or both ends simultaneously. Both pumps have variable feed controls covering their 0-110 gph discharge range. Pump controls have automatic-manual capability. Wetted parts are of materials resistent to alum and ferric chloride.

Two 1200 gallon fiberglass polymer storage tanks are provided: one near the plant influent sewer and the other near the final clarifier. Both have feed pumps similar to the pair at the coagulant tank. In addition, polymer tanks have eductor assemblies for dispersing polymer, and 3 hp mixers for blending fresh batches of polyelectrolytes.

The junction box preceding the final clarifier was modified to provide flash mixing of coagulant. The change involved baffling off a section and installing a mixer there to promote rapid dispersal of the metal salts injected. The arrangement is shown in the following isometric view.

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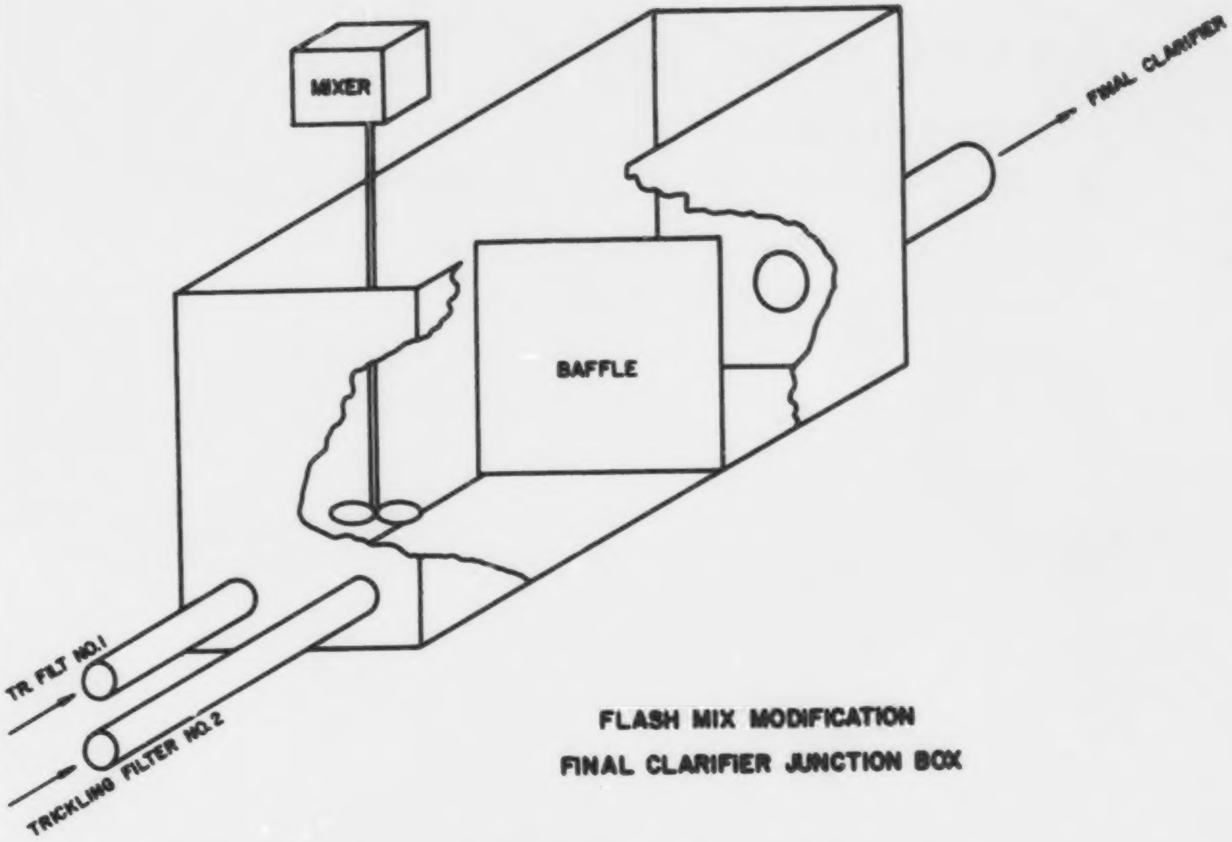


FLASH MIX MODIFICATION
FINAL CLARIFIER JUNCTION BOX

The mixer delivers 2.2 water hp. The approximate velocity gradient for this system is G equals 650/sec. At the average flow of 1.6 mgd, detention is some 50 sec, so Gt equals 32,000.

Further analysis of this dispersion-flocculation system gives information on the reaction times available for flash mixing, intense flocculation, and gentle flocculation. These detention periods are based on simple displacement through: mixing chamber, pipeline, and that part of the centerwell area in the clarifier where active flocculation is observed.

Volumes involved, in gallons:

Flash Mix	1,000
High Energy Flocculation	4,000
Low Energy Flocculation	20,000
TOTAL VOLUME	25,000 Gal

Flow Rates		Coagulation	Flocculation Time (Minutes)		
MGD	GPM	(Minutes)	High Energy	Low Energy	Total
1	700	1.42	5.71	28.6	34.3
1.5	1,050	0.95	3.81	19.1	22.8
2	1,400	0.71	2.86	14.3	17.1
2.5	1,750	0.57	2.28	11.4	13.7
3	2,100	0.48	1.91	9.5	11.4

Flash mixing of coagulants added to raw sewage uses kinetic energy of the turbulant flow entering the wet well. Chemicals are injected at a manhole; mixing begins in a ten feet section of steeply descending sewer which carries plant inflow to the wet well, and dispersal is completed in a confined receiving zone in the wet well.

After a brief (and indeterminant) stay in the wet well, flow is picked up by pumps and passes into the splitter box. Detention time is short and energy levels are fairly high from pumps to clarifiers. All this is followed by flocculation in the centerwell area of the clarifiers, but an estimate of the size of the flocculation zones there has not been possible.

Summarizing, coagulant dispersal and flocculation in raw sewage takes place at ill-defined energy levels and reaction periods. The arrangement is probably not as effective as that in the final effluent, but there has been an attempt to make the best of the existing situation.

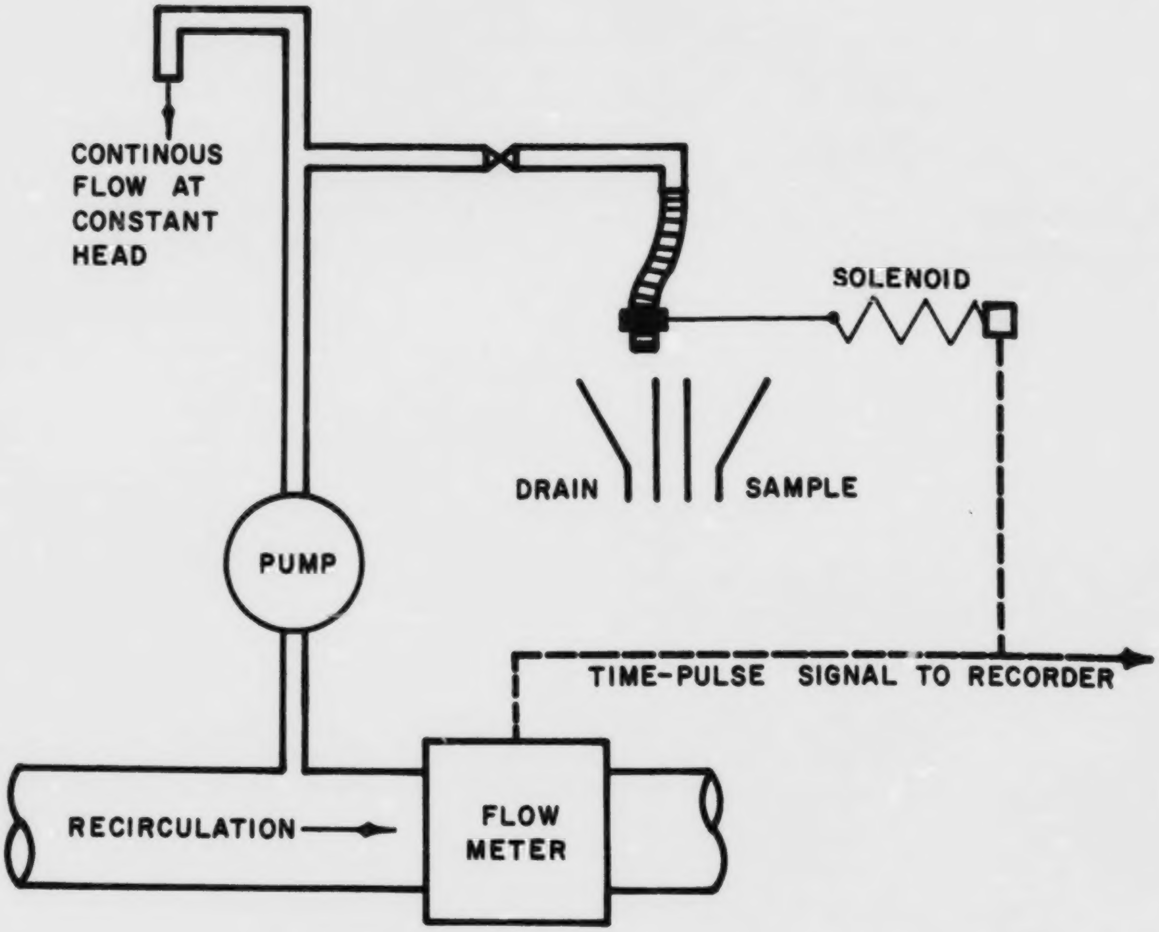
At both ends of the plant, polymer injection facilities deliver into the high energy flocculation zones. Polymers are mixed with 20 gpm carriage water and jetted into a hydraulic regime where there is sufficient turbulence to promote dispersal. A two minute lag time is intended between injection of coagulant and addition of polymers.

IMPROVEMENTS IN FLOW CONTROL AND SAMPLING

One primary clarifier was found to suffer poor inlet hydraulics due to its piping arrangement into the centerwell skirt. A combination splitter-deflector was fabricated and installed to redirect inflow; a good approximation of a velocity dissipating centerwell was obtained.

There were some inaccuracies in the control of flow division in the splitter box preceding the trickling filters. This had been regulated by manual adjustment of gate valves. Proportional weirs were fabricated and installed in the box, insuring an accurate division at all rates of flow.

Recirculation (which includes final sludge) had not been sampled and tested previously. It flows from the bottom of the clarifier, through a gravity line, to the raw sewage wet well. There is a vault housing a flow meter and an air-operated throttling valve at the midpoint of the line. Facilities for automatic sampling were installed in the vault, and function per the following sketch.


CONTINOUS FLOW AT CONSTANT HEAD
SOLENOID
DRAIN
SAMPLE
PUMP
TIME-PULSE SIGNAL TO RECORDER
RECIRCULATION
FLOW METER

RECIRCULATION SAMPLER

A continuous sample flow is withdrawn and split between a constant head shunt and a sample shunt which normally is directed to drain. When the flow meter generates a signal indicating flow in the recirculation line, that signal energizes a solenoid which diverts sample flow to a receiving can. The amount of sample caught is proportional to the amount of recirculation flow.

Two magnetic flow meters were added to the supernatant treatment system: one measures raw flow coming in, and the other measures treated supernatant going to the head of the plant. The difference between their cumulative readings gives the volume of precipitated sludge diverted to drying beds.

The throttling valve on the plant recirculation line had, for years, been controlled by water level in the wet well. Recirculation occurred on a demand basis. For reasons discussed later it became necessary to sharply reduce this flow. An electric timer was wired into the control circuit in a manner which allowed it to override other signals. This timer was eventually set to trigger a 25 second flushing flow every 20 minutes; this pattern established a 70,000 gpd recirculation rate.

Underdrain facilities at the sludge drying beds collect filtrate from wet sludge. Underflow from seven beds comes to either of two filtrate manholes. The manholes were partially dammed and a sump pump was placed in each. A standard water meter was put into the discharge line of each pump, and this permits a record of bed drainage.

A static head of nearly 15 feet of water is available to push digested sludge from digesters to drying beds. This gravity arrangement appears entirely suitable from an operational standpoint. For this project, however, an accurate measure of sludge flow is needed. A piston type sludge pump, with attached stroke counter, was installed for that reason.

MISCELLANEOUS IMPROVEMENTS

A small manually adjusted chlorinator had served the plant adequately for some years. It was replaced by an automatic 2000 ppd unit with compound-loop automatic controls. This changeout was made more to support the demonstration project than it was to modify the plant for chemical addition. Also, after trial operations the waste flow sensing leg was disconnected in the compound-loop control; chlorine flow can be closely controlled by the automatic residual analyzer.

A recording pH meter was installed to monitor plant influent, and a recording dissolved oxygen meter was set up on the effluent. Both of these supply information valuable to the demonstration project, but neither would be really necessary in a normal plant modification.

Ten small pipelines, mostly PVC, were installed at various locations around the plant. These deliver coagulant, polymer dilution water, diluted polymer, rinse water, and sample flows.

Prior to the project, all three digesters were drained and cleaned. Steel stirring mechanisms were strengthened or rebuilt as necessary. This insured that the digestion system was free of grit and was mechanically functional for the study.

LABORATORY FACILITIES AND PERSONNEL

The existing plant had its control laboratory located in the main building. The arrangement was somewhat cramped, so a separate lab building was built just prior to beginning the project. Additional space and personnel were added to support the study.

There are thirteen sampling points in the treatment plant, and up to twenty-three analyses may be run on the composite samples taken. Appendix B shows the stations and analyses involved. Approximately 140 items of data are reported on each sample day.

A staff of three analysts works five days per week, usually Monday through Friday. The samples they run are those collected Sunday through Thursday. Occasionally they work on weekends to gain data on samples collected Friday and Saturday. A fourth analyst, who works about 30 hours a week, helps handle the heavy lab load involved in the project.

Lab effort related solely to the project includes a statistical quality control program to verify data, zeta and jar test studies on coagulants and polymers, total and fecal coliform tests, turbidity, COD, and a broad range of analyses to trace buildup or removal of particular substances.

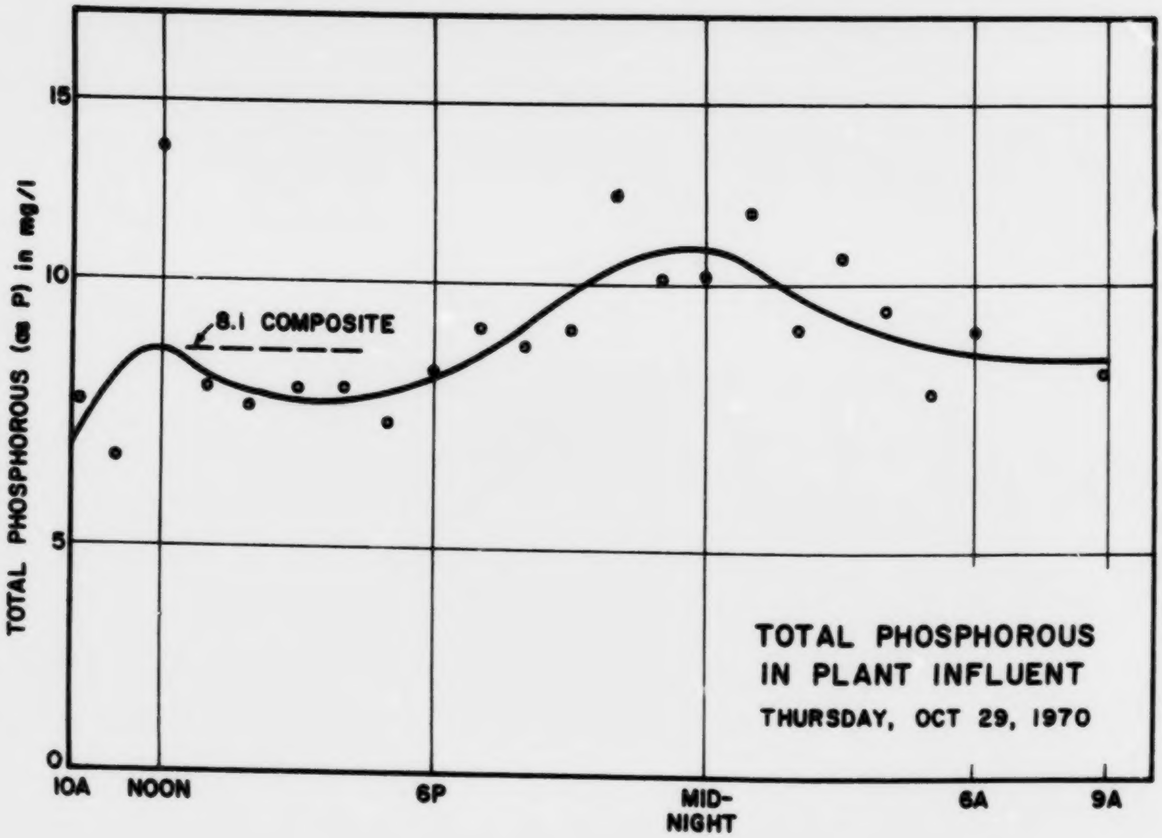
From a strict standpoint of normal operational control of chemical precipitation, the typical tests for a conventional plant (BOD, solids, pH, temperature, dissolved oxygen) should be supplemented by phosphorous, alkalinity, and perhaps elements involved in the coagulant (iron or aluminum, and sulfate or chloride). Some phosphorous tests should be made on hourly grab samples, as discussed later. The other tests would be run on daily composites. Also, jar tests would be needed to guide determination of proper chemical feed; these would be similar to those required in a water treatment plant.

A final category of tests has been run routinely for some time: various nitrogen forms and sulfides. Whether these are required for operational control at other plants would be a matter of individual judgement.

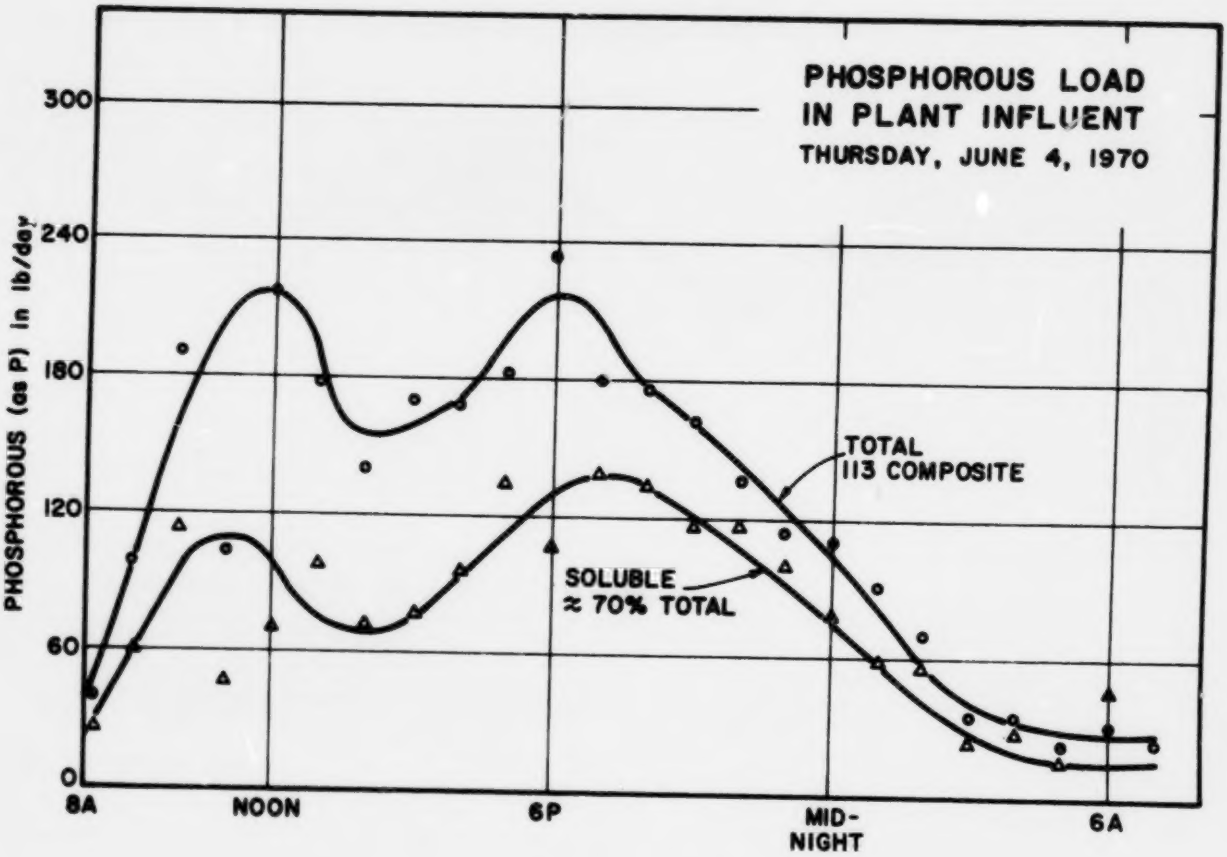
PRELIMINARY OPERATIONAL RESULTS

Liquid alum was added during waste treatment in the fall of 1970. Considerable data were taken and are still being analyzed. This information has scant value until the study is finished and put into perspective. It seemed proper to share some of the data, with the understanding that THESE ARE PRELIMINARY OBSERVATIONS---NOT DEFINITIVE CONCLUSIONS. Readers should appreciate this point and temper their views accordingly.

To begin, two illustrations show typical values of phosphorous coming into the plant. The first plots phosphorous concentration based on hourly samples.

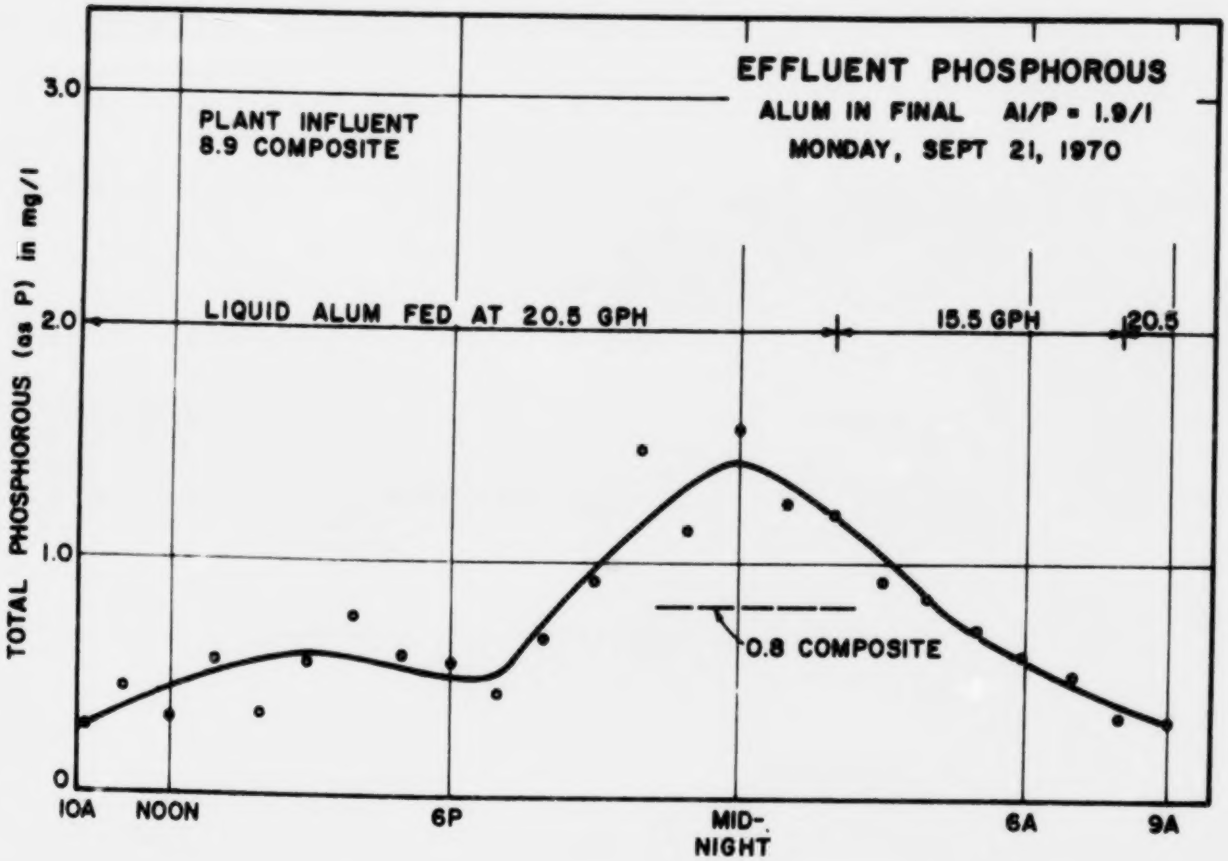

TOTAL PHOSPHOROUS (as P) in mg/l
15
10
5
0
8.1 COMPOSITE
10A
NOON
6P
MID-NIGHT
6A
9A
TOTAL PHOSPHOROUS
IN PLANT INFLUENT
THURSDAY, OCT 29, 1970

Composite phosphorous income (as P) was 8.1 mg/l, slightly less than the statistical mean. The next plot shows typical phosphorous load (lb/day) and shows the soluble fraction.

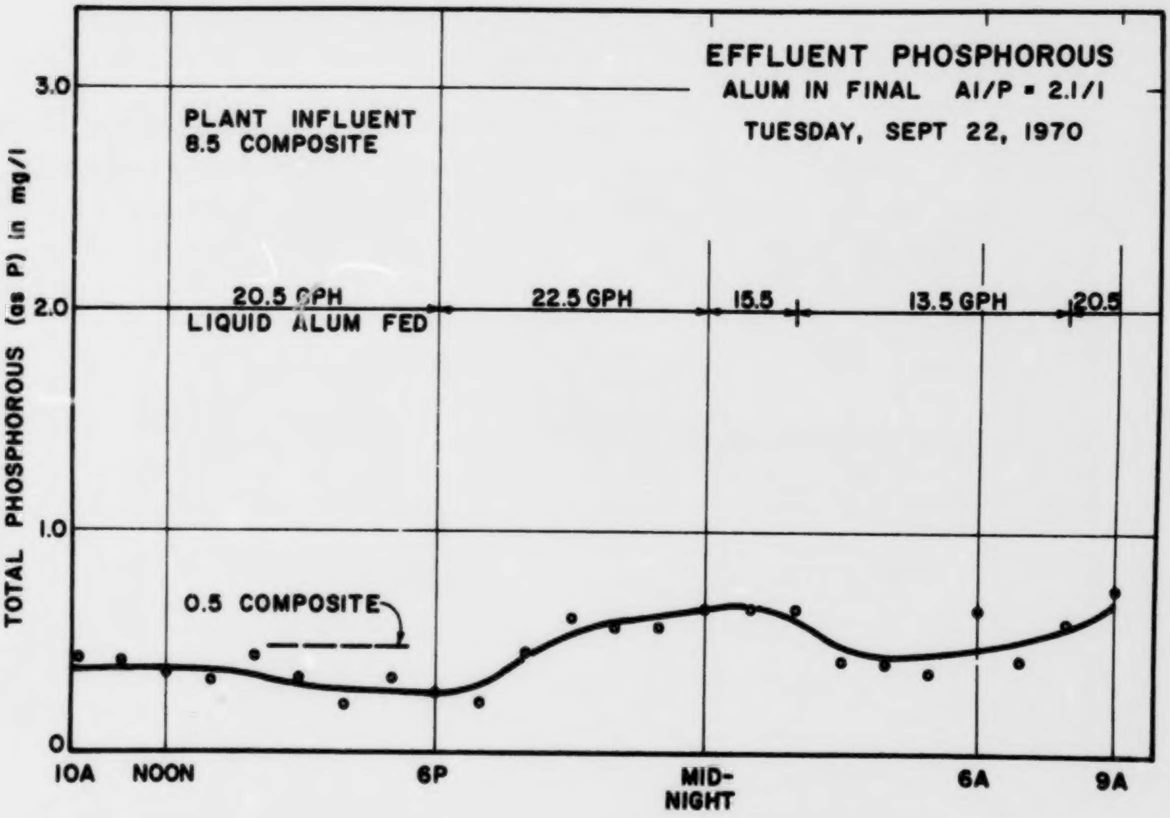


Total phosphorous in the composite was 113 lb/day. The soluble fraction was 70%, a ratio which is believed consistent.

During September liquid alum was added to flow entering the final clarifier. After a period of fixing leaks in chemical delivery pipes, recalibrating chemical pumps, and other startup activities, some proper results developed. Two key items were learned: hourly effluent grab samples are vital to regulation of feed rates, and low recircultation rates (70,000 gpd) are necessary to preserve the floc blanket which develops around the centerwell. When the blanket is present, effluent is clear and low in phosphorous, BOD, and suspended solids. When the blanket is absent, results suffer. The next plot shows performance during a day when two chemical pump settings were used.



Notice the mole ratio of aluminum to total phosphorous was 1.9 to 1.0. Phosphorous levels in plant inflow were normal. Effluent phosphorous got out of control near midnight, and the mean effluent concentration that day was 0.8 mg/l. This performance led to changing chemical feed rate five times per day, and resulting improvements are shown in the next graph.

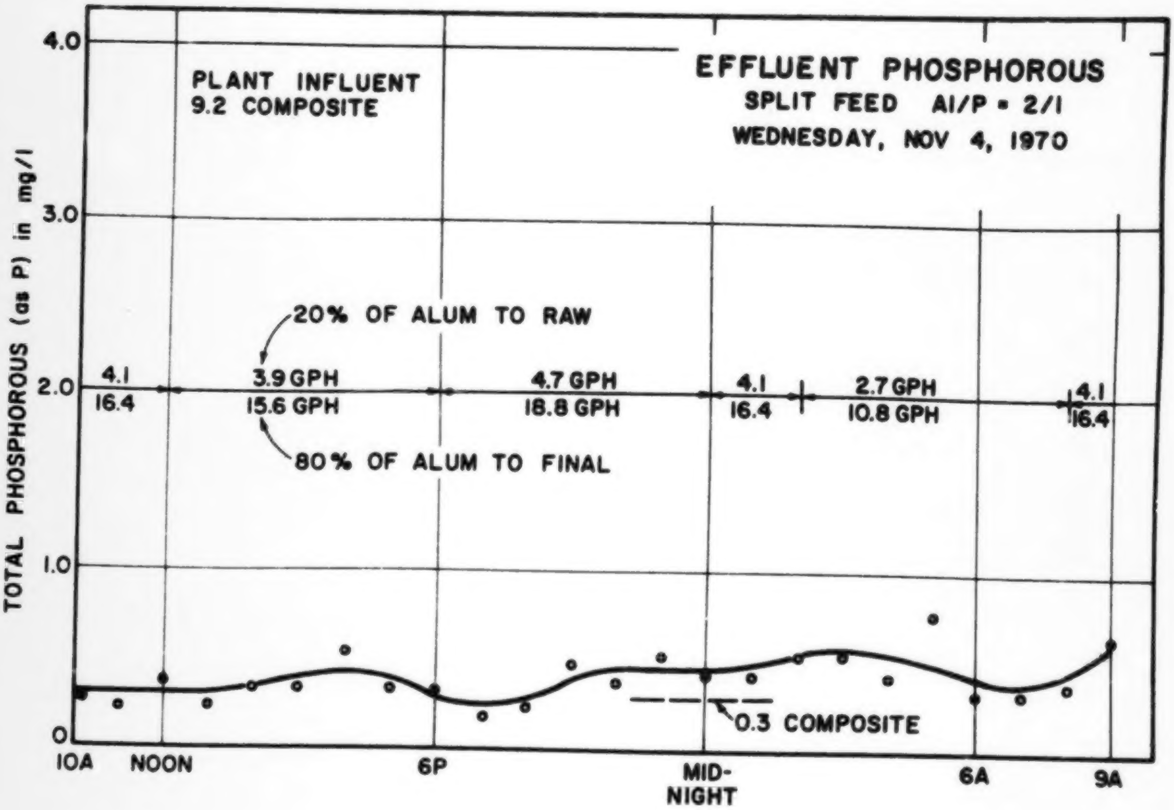


In this case the mole ratio was 2.1 to 1.0. There were no sharp peaks in effluent phosphorous levels, and the composite sample showed 0.5 mg/l escaped.

At this time it is felt that multiple settings of chemical pumps are vital to good performance with minimum alum consumption.

In October alum was added to raw sewage. Performance seemed not as good as before, and signs of digester overloading led to termination after nine days of trial. No data or further comments can be offered at this point.

In November split feeding (to raw and final) was tried. The following figure shows results of one of the better days.



Feed was 20% to raw and 80% to final. The mole ratio was 2 to 1. Composite samples showed phosphorous was 9.2 mg/l in plant influent and 0.3 mg/l in the effluent.

The preceding plots illustrate performance on some of the better days to date; there are indications that good results can be gotten day after day, but this point lacks proof. Further, it has not been proven that digesters can accept increased loads (from any of the feed regimes) on a long term basis. The changes in character of digested sludge have not been properly studied. Much data already collected is not discussed here. More data is needed. The only point made in this section is that the precipitation facilities which have been used so far appear to perform mechanically, and the overall approach holds some degree of hope for reliable performance.

COSTS OF FACILITIES AND OPERATIONS

Part of the Richardson treatment plant was built in 1953 at a total cost of $75,000, exclusive of land. In 1961, treatment facilities were enlarged to the present arrangement; the total cost of those improvements was $0.25 million, excepting land.

A new laboratory building was added in 1969. This facility is considered an integral part of the treatment system, but perhaps one-third of its cost went to extra space for the demonstration project. The building was built at a total cost of $33,000.

The laboratory was outfitted at a cost of $11,000. Furniture accounted for $2,000 of the total, and the $9,000 balance went for equipment and supplies.

Some of the equipment might not be required except for the demonstration project: a $3,000 zeta meter, a $600 recording pH meter to monitor plant inflow, a $700 recording dissolved oxygen meter to monitor effluent, and some $700 worth of special glassware and chemicals. The remaining $4,000 would have been spent for laboratory facilities to support chemical coagulation in the treatment plant; major items include $800 for an advanced type of jar test apparatus, an $800 analytical balance, and a $500 spectrophotometer.

If the laboratory had been built solely to support chemical coagulation in the treatment plant, it would have cost about $21,000 for the building and $6,000 for furniture, equipment, and apparatus.

Modifications to the treatment plant came to a total cost of $53,000 distributed as follows:

Materials	$35,000
Labor	7,000
Supervision	3,500
Design and Misc.	7,500
TOTAL	$53,000

Construction labor and supervision were by city personnel, and figures cited include a factor for overhead. The high ratio of materials to labor relates to such expensive equipment as a $6,800 automatic chlorinator, two magnetic flow meters at a $4,200 cost, three fiberglass chemical tanks at $5,000 total, two polymer mixers at $2,200 total, and four chemical feed pumps at $6,800 total. All these items total $25,000 which is considerably more than it cost to install them.

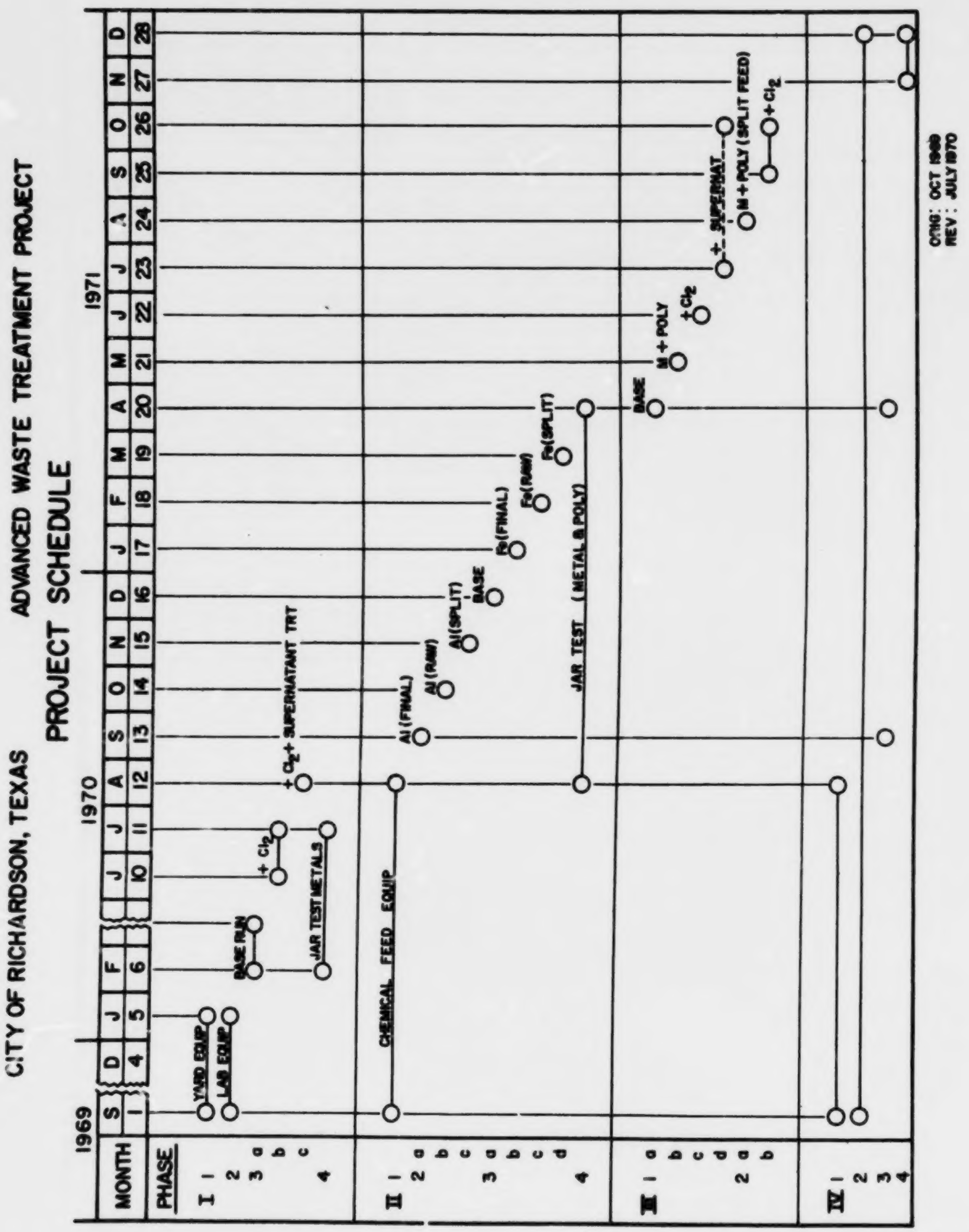
Combining all facility costs related to providing chemical coagulation, a total outlay of $80,000 was required. This converts to $50,000 per MGD of plant capacity, but that unit cost figure could not be extended to estimate costs at other plants. For example, perhaps $60,000 would be required to outfit a 0.5 MGD plant, and $100,000 might outfit a 5 MGD plant. In other words, the basic equipment would cost about the same no matter what size plant is involved.

Operating costs for chemical coagulation consist almost entirely of chemical costs alone. Additional power might cost $200 or $300 per year. The existing operating staff can run the equipment. Control testing might require an additional analyst in the laboratory, but insufficient experience is available to verify this. Liquid alum costs 33¢ per pound of aluminum, delivered in Richardson. Making some broad assumptions regarding demand, alum costs might run 4¢ to 5¢ per 1000 gallons of water. This would total some $25,000 per year at this plant. No information is available now regarding efficiency or cost of ferric iron as a coagulant, nor has the role of polymers been studied.

In summary, construction costs for chemical coagulation are well defined for the Richardson facilities, but they might be quite different at another plant. It appears most of the operating costs would go to purchase coagulant, and some highly generalized figures have been offered to give some sort of perspective.

APPENDIX A **PROJECT SCHEDULE**

The projected activities are shown on this sheet. A written key on the following page gives more details on abbreviated entries.



City of Richardson, Texas Advanced Waste Treatment Project

SUMMARY OF PROJECT SCHEDULE

Phase	Task	Month	Activity
I	1	1-5	Yard Equipment (Meters, Manholes, Samplers, Cl_2 Equip), Provide & Install
	2	1-5	Lab Equip, Provide & Install
	3a	6-9	Plant Baseline Operation
	b	10-11	Plant Baseline Operation plus Chlorination of Effluent
	c	12	Plant Baseline Operation plus Chlorination and Supernatant Treatment
	4	6-11	Jar Test Metal Salts on Raw
II	1	1-12	Chemical Feed Equip, Provide & Install
	2a	13	Baseline Operation plus Alum Salt in Final
	b	14	Baseline Operation plus Alum Salt in Raw
	c	15	Baseline Operation plus Alum Salt Split Feed
	3a	16	Baseline Operation (Purge & Check)
	b	17	Baseline Operation plus Iron Salt in Final
	c	18	Baseline Operation plus Iron Salt in Raw
	d	19	Baseline Operation plus Iron Salt Split Feed
	4	12-20	Jar Test (Metal Salt + Polymer) on Raw and Filter Effluent
III	1a	20	Baseline Operation (Purge & Check)
	b	21	Baseline Operation plus Salt-Poly Fed at Best Points
	c	22	Baseline Operation plus Salt-Poly plus Chlorination of Effluent
	d	23-26	Baseline Operation plus Salt-Poly plus Chlorination plus Supernatant Treatment
	2a	23-26	Optimize for best performance and economy
IV	1	1-12	Monthly Letter-Reports
	2	1-24	Preparation & Submission of Expense Statements
	3	13&20	Progress Reports (Phase I and II)
	4	27-28	Final Report

Orig: Oct 1969
Rev: Jul 1970

APPENDIX B

DATA MATRIX

This sheet shows major items reported daily. All are calculated and recorded directly on Lab Bench Sheets, then transferred directly onto computer cards. A typical Bench Sheet (for ammonia nitrogen) is shown on the following page.

	RAW	PRIM EFF	FILT EFF	FINAL EFF	RECIRC	SLUDGE 1	SLUDGE 2	SLUDGE 3	RAW SUP 1	RAW SUP 2	RAW SUP 3	TREAT SUP	BED DRAIN	
FLOW	●				●	●	●	●	●	●	●	●	●	
TOT SOL	●	●	●	●	●	●	●	●	●	●	●	●	●	
TOT VOL SOL	●	●	●	●	●	●	●	●	●	●	●	●	●	
SUS SOL	●	●	●	●	●				●	●	●	●	●	
SUS VOL SOL	●	●	●	●	●				●	●	●	●	●	
SET SOL	●	●	●	●										
BOD	●	●		●					●	●	●	●		
DO	●			●										
COD	●	●	●	●	●				●	●	●	●	●	
PHOS	●	●		●	●	●	●	●	●	●	●	●	●	
TKN	●	●		●	●	●	●	●	●	●	●	●	●	
NH3	●	●		●		●	●	●	●	●	●	●	●	
NO2				●										
NO3	●			●										
ALK	●	●		●		●	●	●	●	●	●	●	●	
FE	●	●		●	●	●	●	●	●	●	●	●	●	
AL	●	●		●	●	●	●	●	●	●	●	●	●	
S	●					●	●	●	●	●	●	●		
SO4	●	●	●	●		●	●	●	●	●	●	●	●	
CL	●	●		●										
PH	●	●	●	●		●	●	●	●	●	●	●	●	
TEMP	●	●	●	●		●	●	●	●	●	●	●	●	
TURB	●	●		●										
COLI	●			●										
FECAL	●			●										

PARAMETER | 1 | 4 | NH3
(1 2)

DATE | | | | | |
(3 4 5 6 7)

STATION	ABSORBANCE	FACTOR	MG/SAMPLE	DILUTION	RESULTS	COMPUTER
RAW					— —•— —	• (27) 25 26 27 28
PRIMARY EFFL					— —•— —	• (31) 29 30 31 32
FILTER EFFL						33 34 35 36
FINAL EFFL					— —•— —	• (39) 37 38 39 40
RECIRC						41 42 43 44
SLUDGE 1					— — —•—	• (48) 45 46 47 48
SLUDGE 2					— — —•—	• (52) 49 50 51 52
SLUDGE 3					— — —•—	• (56) 53 54 55 56
RAW SUP 1					— — —•—	• (60) 57 58 59 60
RAW SUP 2					— — —•—	• (64) 61 62 63 64
RAW SUP 3					— — —•—	• (68) 65 66 67 68
TREAT SUP					— — —•—	• (72) 69 70 71 72
BED DRAIN					— — —•—	• (76) 73 74 75 76
STANDARD					— — —•—	

425 mμ

APPENDIX C

DATA PROCESSING

Electronic data processing is used to handle the 140 items of data generated each day. There are two major programs: a daily report, and a statistical report. In the daily report the following items are printed out on one page:

1. Up to 23 parameters at each of 13 sampling stations
2. Five computed loadings at up to 12 stations, expressed in lb/day
3. Two ratios at 10 stations: BOD/COD, and SVS/Susp Solids
4. Six clarifier loadings: hydraulic, and solids
5. Four filter loadings: hydraulic, and organic
6. Two digester loadings: hydraulic, and volatile solids
7. Twelve percentages of removal: primary units, and overall

The statistical report comes from processing selected groups of daily reports, and each one-page printout considers one parameter (e.g. BOD) at all of the sampling stations. Printouts include:

1. Tally of number of occurrences
2. Max-Min values
3. Arithmetic mean, and standard deviation
4. SD/Mean
5. Mean ± SD
6. Mean ± 2SD
7. Five selected cumulative probability tables
    a. Group limits and plotting position
    b. Population within groups
    c. Exceedance within groups (95% gaussian normality)
    d. Cumulative frequency

CHEMICAL PRECIPITATION

Sidney A. Hannah

CHEMICAL PRECIPITATION

Sidney A. Hannah

1. Historic

The recent interest in chemical precipitation for treatment of wastewater was preceded by a similar interest some sixty years ago. The Royal Commission of London (24) issued a report which endorsed the practice. The process was intended to replace primary sedimentation. Alum or lime was used to flocculate raw sewage to produce effluents with 10-40 mg/l of suspended solids. The practice was not widely adopted, however, because of the engineering difficulties and particularly because of the problems of handling the sludge.

The Guggenheim process proposed in the thirties also involved chemical precipitation but this also failed to be widely adopted. Possibly, a dominant reason for the failure of adopting chemical precipitation at that time, aside from the extra cost, engineering and sludge problems, was the lack of pressure to produce a better effluent.

2. Objectives of Chemical Precipitation

The impetus to use chemical precipitation is provided by the need to remove phosphorus. But chemical precipitation, while removing phosphorus, also removes in excess of 95 percent of the suspended solids yielding a fully clarified effluent with 95 percent of phosphorus removed or in precipitated form.

Conventional treatment techniques of settling and biological oxidation are capable of removing some phosphorus as shown in Fig. 1. In spite of some encouragement that certain plants are capable of greater removals, the most that can be expected in general are 5-15 percent by sedimentation, 20-30 percent through a trickling filter and up to 50 percent by activated sludge. Moreover, actual phosphorus removal will ultimately depend on the amount of sludge permanently removed from the process stream. This means that phosphorus concentrated in the sludge must be removed from digester contents.

FIGURE 1 Phosphate Removal by Conventional Waste Treatment Plants

Type of Treatment Plant	% Phosphorus Removal
Primary Sedimentation	5-15
Primary and Trickling Filter	20-30
Primary and Activated Sludge	30-50

The most economic and reliable method of removing phosphorus and suspended solids is chemical precipitation. The choice of point of addition depends on several factors which are shown in Fig. 2. Chemicals may be added to raw sewage with separation of solids occurring in the primary, to the aerators with separation obtained with the biological sludge, or chemicals may be added to the final clarifier as a tertiary process. All of these variations have been studied and are technically feasible.

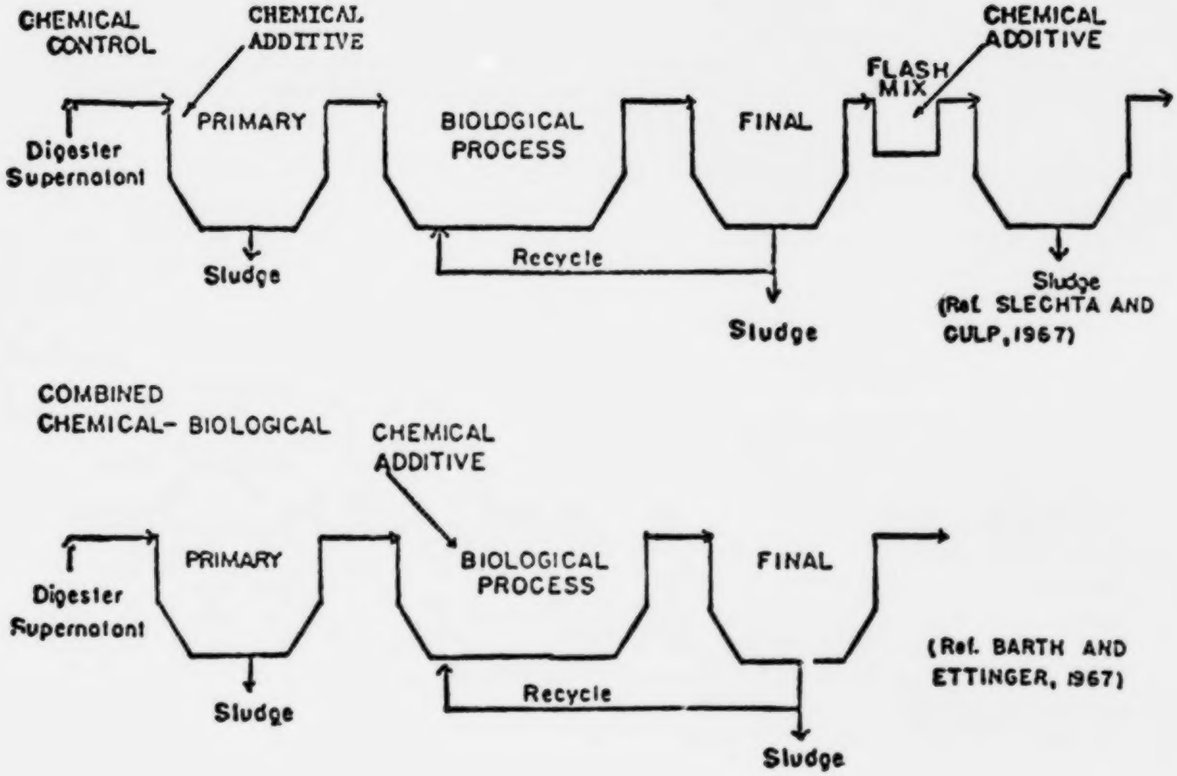
A choice also has to be made on the chemical to use. The factors to be considered here are shown in Figure 3. The influent phosphorus level and particularly the permissible residual phosphorus are important considerations. Wastewater characteristics determine in part the chemical dosage and finally, facilities for sludge handling and ultimate disposal of the sludge must be considered.

A variety of chemicals can be used for chemical precipitation. These are shown in Fig. 4. Various salts of iron are equally effective. These salts include ferrous chloride or ferrous sulfate, ferric chloride or ferric sulfate, and waste pickle liquor which is a waste product from steel processing and consists of largely ferrous chloride or sulfate with some ferric salts. Lime or sodium hydroxide must be added with the ferrous salts. The pH of optimum precipitation with the latter appears to be around pH 8.0.

Aluminum salts may also be used. Commercial alum, aluminum sulfate, is the more common although sodium aluminate has also been used. The latter, while more expensive, has the advantages of containing some excess caustic which helps to maintain pH and does not add sulfate ion, as alum does.

Lime as a precipitant is very attractive because of its cheap initial cost and because sludge can be calcined to recover reuseable lime while at the same time incinerating the organic sludge.

All of the metal precipitants may require the addition of polymer to obtain effective settling and separation of solid from liquid.


CHEMICAL CONTROL
CHEMICAL ADDITIVE
Digester Supernatant
PRIMARY
BIOLOGICAL PROCESS
FINAL
FLASH MIX
CHEMICAL ADDITIVE
Sludge
Recycle
Sludge
Sludge
COMBINED CHEMICAL- BIOLOGICAL
CHEMICAL ADDITIVE
Digester Supernatant
PRIMARY
BIOLOGICAL PROCESS
FINAL
Sludge
Recycle
Sludge

(Ref. SLECHTA AND CULP, 1967)

(Ref. BARTH AND ETTINGER, 1967)

Figure 2 PHOSPHORUS CONTROL BASIC SYSTEMS

Figure 3

FACTORS AFFECTING CHOICE OF CHEMICALS

1. INFLUENT PHOSPHORUS LEVEL
2. EFFLUENT DISCHARGE STANDARD
3. WASTEWATER CHARACTERISTICS
4. PLANT SIZE
5. CHEMICAL COSTS INCLUDING TRANSPORTATION
6. SLUDGE HANDLING FACILITIES
7. SLUDGE DISPOSAL FACILITIES
8. OTHER PROCESSES UTILIZED

Figure 4 Alternative Chemical Systems for Phosphorus Removal in the Primary

---

A. Iron

Ferrous Chloride or Sulfate - Base - W/O* Polymer

Commercial or Waste Pickle Liquor | Lime or Sodium Hydroxide

Ferric Chloride or Sulfate - W/O Polymer

B. Aluminum

Alum or Aluminate - W/O Polymer

C. Lime - W/O Additives

1 or 2 Stage

*W/O - With and Without

---

3. Performance Results

(a) Precipitation with iron salts

All of the chemicals mentioned and all of the points of addition have been tested at full scale. These tests provide the information for any plant to determine the choice of chemical and point of addition using the factors pointed out in Fig. 2 as the basis for decision.

The addition of metallic salts to primary clarifiers not only removes phosphate but obtains clarification thus providing suspended solids, BOD-COD and phosphate removal all at the same time. Thus, the organic load to the secondary system is reduced with possible reduction in air requirements and waste activated sludge production.

The results of full-scale chemical treatment at Grayling, Michigan, are shown in Figure 5. Without chemicals, removal in the primary amounted to 50% for SS and 41% for BOD. No removal is reported for phosphorus, although I suspect that some removal was obtained. Adding 15-30 mg/l of iron as ferrous chloride increased SS removal to 78 percent, BOD removal to 58 percent and 72 percent of the phosphate was precipitated. Note that with ferrous salts, sodium hydroxide at the rate of 30-50 mg/l as $CaCO_3$ had to be added. Most chemical precipitation requires polymer for effective clarification. In this case 0.3-0.5 mg/l was used.

A similar use of ferrous salt was tried at Mentor, Ohio, Fig. 6. In this case waste pickle liquor was used as the source for ferrous chloride. Waste pickle liquor was added at a dosage to yield 43 mg/l as Fe. Lime, rather than caustic, was added at the rate of 66 mg/l. Again polymer was required - 0.4 mg/l. Removals of SS and BOD were similar to the Grayling test but phosphate removal was superior amounting to 83.5 percent. The greater phosphate removal can be accounted for by the greater iron dosage. Removals in excess of 90-95 percent can be obtained with even greater iron dosages and careful separation of the precipitated phosphate. The two forms of pickle liquor are apparently interchangeable. Ferrous sulfate was successfully used at Texas City. When available within reasonable hauling distances, waste pickle liquor containing 6-9 percent of ferrous iron can be an inexpensive source for iron. The hauling costs amounted to 1.5¢/lb of Fe at Texas City and 2¢/lb Fe at Mentor. The cost of lime or sodium hydroxide must be considered. Also, the ferrous iron must be oxidized to ferric form to obtain removal of both phosphate and iron, hence sufficient oxygen must be present. Theoretical work with ferrous iron in a contract with Atomics International has shown that the optimum pH centers around pH 8.0.

The need to control pH and to ensure availability of oxygen to prevent excessive soluble ferrous compounds in the effluent has tended to diminish

Figure 5 RESULTS AT GRAYLING, MICHIGAN

CHEMICALS:

$FeCl_2$ = 15-30 mg/l as Fe

NaOH = 30-50 mg/l as $CaCO_3$

A-23 = 0.3-0.5 mg/l

	WITHOUT CHEMICALS (10 weeks)			WITH CHEMICALS (10 weeks)		
	Raw mg/l	Pri.Eff. mg/l	% Rem.	Raw mg/l	Pri.Eff. mg/l	% Rem.
SS	157	78	50	224	50	78
BOD	170	101	41	178	74	58
TOTAL P	No removal			15.5	4.4	72

Figure 6 RESULTS AT MENTOR, OHIO

CHEMICALS:

WASTE PICKLE LIQUOR ($FeCl_2$) = 43 mg/l as Fe

LIME = 66 mg/l

A-23 = 0.4 mg/l

PERCENT REMOVAL WITH CHEMICALS

SS = 74%

BOD = 59%

TOTAL P = 83.5%

the advantages of the low cost ferrous iron. For these reasons ferric chloride has been more widely accepted. Full-scale testing with ferric chloride and polymer in the primary is being conducted at Grand Rapids, Michigan, at Benton Harbor, Michigan, and elsewhere.

Addition of chemicals to the primary and increased capture of organic solids clearly indicate that more sludge solids will have to be handled. Good quantitative data on this aspect are lacking although the study at Grand Rapids should be informative. Some data were obtained at Benton Harbor, Michigan, Fig. 7. The study was conducted in three stages; a baseline period with no chemical addition; a second period when ferric chloride and polymer were added but with wasting of activated sludge to the primary, and a third period when chemicals were added but waste activated sludge was not cycled to the primary. The plant uses the "Kraus Process" which involves returning a reaerated portion of the waste activated solids to the primary.

Chemical addition actually reduced the volume of sludge pumped but the solids content increased from 3.8 to 4.82 percent with chemical addition and no recycle. With increased organic removal, dissolved oxygen in the aeration tank increased from 4.8 to 6.6 mg/l. This benefit is obscured by the increased air supplied to the tank.

The presence of chemical sludge posed no problem in the digesters. With increased organic solids pumped to the digester, total gas production increased. Gas production per pound of solids also increased from 3.48 ft^3/lb to 3.67 ft^3/lb. Operation was otherwise normal. A point worth mentioning here is that soluble phosphate concentration in the digester supernatant return flow was very low.

(b) Precipitation with aluminum salts

Either alum or sodium aluminate can be used for precipitation in the primary but has been little used for this purpose. Aluminum salts have been used as a chemical precipitant in the aerator. This aspect is discussed by another speaker. One reason why alum has not been so widely used in raw sewage clarification is its higher cost over iron salts. Sludge from iron precipitations are easier to handle than alum sludges. Alum, nevertheless, is widely used in Sweden in raw sewage flocculation because of the much lower cost of commercial alum in Sweden.

If very high quality effluent is required for discharge or reuse, treatment of the final effluent or tertiary treatment may be necessary. Both iron or aluminum salts can produce an effluent low in phosphorus, BOD and suspended solids. Alum, rather than iron salts, was chosen for the

Figure 7 Plant Data at Benton Harbor Using Ferric Chloride

	No Chemical Addition	Chemical Addition	
		Recycle to Primary*	No Recycle*
Primary Clarification			
Volume Pumped, Gal/Day	53,200	51,600	46,100
Solids Concentration,	3.80	4.29	4.82
Percent Volatile	64	63	60
Mixed Liquor Data			
Suspended Solids, mg/l	2500	2700	2400
Sludge Volume Index	85	78	64
Dissolved Oxygen	4.8	5.9	6.6
Air Applied, cfm	4480	4940	4710
Digester Operation			
Gas Production - ft^3	37,800	44,400	40,800
Gas Production - ft^3/lb of solids	3.48	3.82	3.67

*With and without Recycling of Waste Activated Sludge

400 gpm plant at Nassau County, New York. The objective of this study is to determine the feasibility of treating secondary effluent to a quality suitable for ground water discharge. In this instance, iron could not be used. The processes include, alum clarification, dual media filtration, and adsorption on granular activated carbon. Fig. 8 shows results of operation of the clarification and filtration portions of the systems. With an average alum dose of 200 mg/l, phosphorus was reduced to 1.4 mg/l in the clarifier effluent. It is significant that filtration produced a highly clarified effluent containing 0.08 mg/l P and 0.2 JTU turbidity. The filter served to capture the precipitated phosphates that did not settle in the clarifier. An important observation is that when sufficient chemical is added to obtain good clarification, good phosphorus removal is also obtained.

Figure 8 NASSAU COUNTY OPERATING RESULTS

CONSTITUENT	INFLUENT	CLARIFIER EFFLUENT	FILTER EFFLUENT
POLYPHOSPHATE, mg/l P	0.5	-	-
TOTAL PHOSPHATE, mg/l P	7.2	1.4	0.08
TURBIDITY, JTU	50	6	0.2

The quality of secondary effluents can be considerably increased by chemical precipitation and sand filtration. Some work done by the Water Pollution Research Laboratory in England on two treatment plants is summarized in Fig. 9. All parameters of product quality were substantially reduced. Final effluent contained 2-5 SS, 2-3 BOD, 1.5-0.15 phosphate and turbidity 0.8 to 3.5 units. Soluble aluminum ranged 0.1 to 0.45 mg/l.

FIGURE 9
TREATMENT OF EFFLUENTS OF POOR QUALITY
BY CHEMICAL COAGULATION AND SAND FILTRATION

mg/l unless otherwise stated	Effluent A		Effluent B	
	Initial	Final	Initial	Final
SUSPENDED SOLIDS	84	2	26	5
BOD	35	2	25	3
COD			83	26
PHOSPHATE	11	1.5	2.5	0.15
ALUMINUM		0.1		0.45
COLOR (HAZEN UNITS)	90	33	42	27
TURBIDITY (ATU)		0.8		3.5

(c) Lime precipitation

The chemistry of lime precipitation is entirely different from the chemistry of iron or aluminum. With the latter the dose is related to phosphate content and the requirement to provide sufficient hydrolysis products of the metal. In practice these requirements have been met with dosages of alum or iron ranging from 150 to 300 mg/l. These dosages will generally produce a clarified effluent low in phosphorus. Lime dosage, on the other hand, is more dependent on the wastewater characteristics, principally alkalinity and hardness. In practical terms the dosage is determined by the pH required to obtain the desired clarification and phosphorus removal. When slaked lime $Ca(OH)_2$ is added to wastewater, it reacts with alkalinity precipitating calcium carbonate; it reacts with orthophosphate to precipitate hydroxyapatite, $Ca_5(OH)(PO_4)_3$. Precipitation of magnesium hydroxide begins around pH 10 and is complete at pH 12.

The amount of lime required to raise the pH to a given value is a function of the initial alkalinity. This relationship is shown in Fig. 10. Wastewaters with high alkalinity (buffer capacity) necessarily require high doses to reach a pH range where phosphorus precipitation is effective. The higher the alkalinity, the greater the lime requirement. The pH range that must be obtained for high phosphorus removal is about pH 10 to 11. At these pH's residual total phosphorus will range around 0.1 to 0.3 mg/l. This relationship is shown in Fig. 11. Looking at Fig. 10 again, the lime dose to obtain pH 10 can range from as little as 50-75 mg/l for a wastewater with an alkalinity of 22 mg/l to almost 400 if the alkalinity is 600 mg/l.

The two extremes of lime clarification in high and low alkalinity waters can be illustrated by the operation of the pilot plants at Lebanon, Ohio, and Blue Plains at Washington, D.C. A pilot plant at Lebanon, Ohio, treats secondary effluent with lime and media filtration. This wastewater has a relatively high hardness and alkalinity. Operation at pH 10.7 resulted in a phosphorus concentration of 0.5 mg/l and 0.1 mg/l in the clarifier and filter effluents respectively. The lime requirement has ranged from 300-400 mg/l. Because of the relatively high hardness at Lebanon, a sufficient portion of the sludge consisted of dense calcium carbonate which settles readily, thus the entire operation can be conducted in a single stage.

Lime clarification at the Blue Plains pilot plant where the wastewater is low in both hardness and alkalinity must be operated either as a two-stage system or a single-stage with the addition of carbonate as soda ash or CO_2. There is insufficient alkalinity to permit the formation of a dense, good settling calcium carbonate sludge with lime addition alone. The most effective treatment sequence has been a two-stage process; lime to pH 11.5 in the first-stage clarifier, reduction of pH to 9.5-10.0 by addition of CO_2 and clarification in the second stage with the aid of 6-12 mg/l Fe^{+++}. Results obtained with single-stage

FIGURE 10

ALKALINITY, LIME DOSE, AND pH.

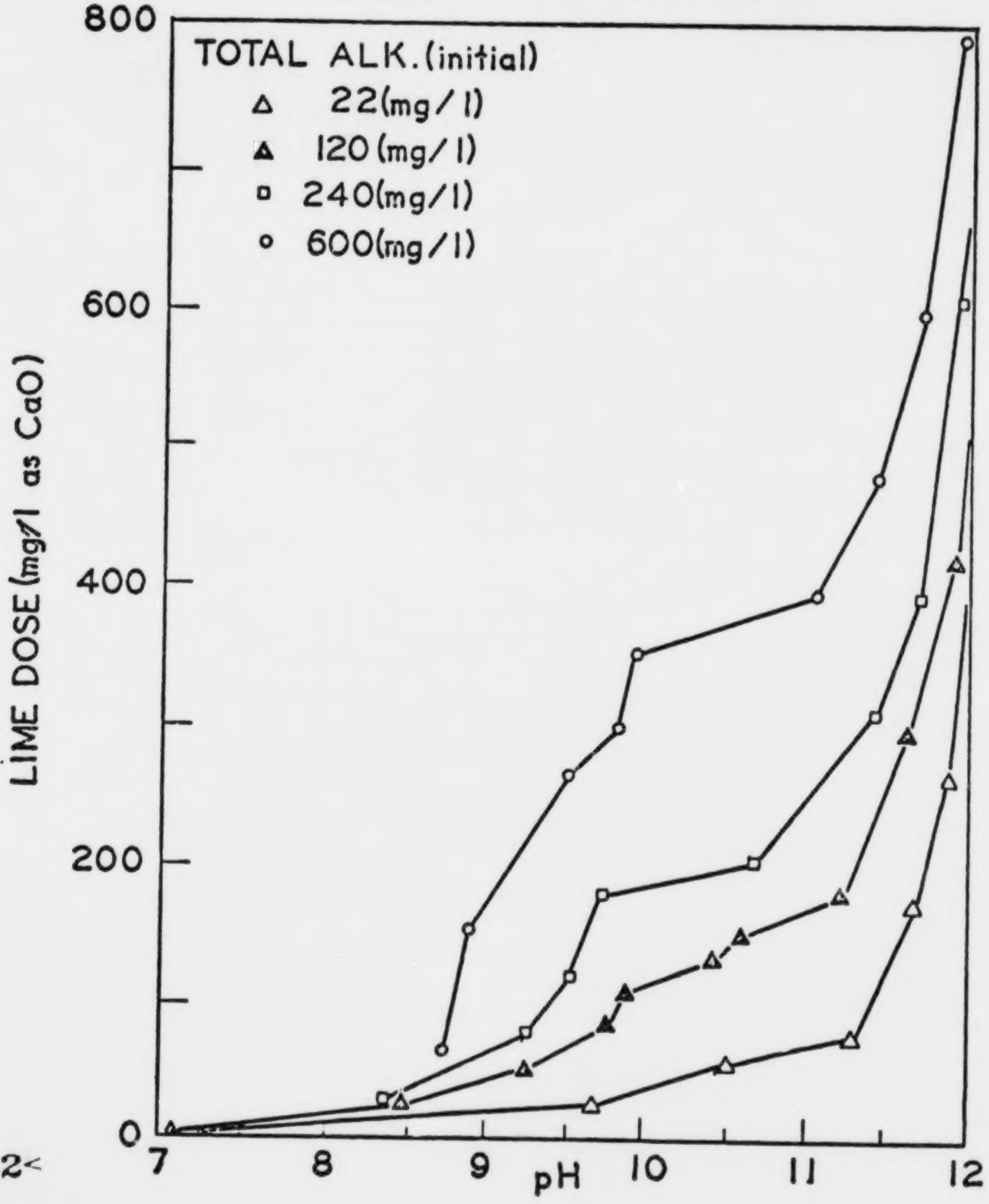


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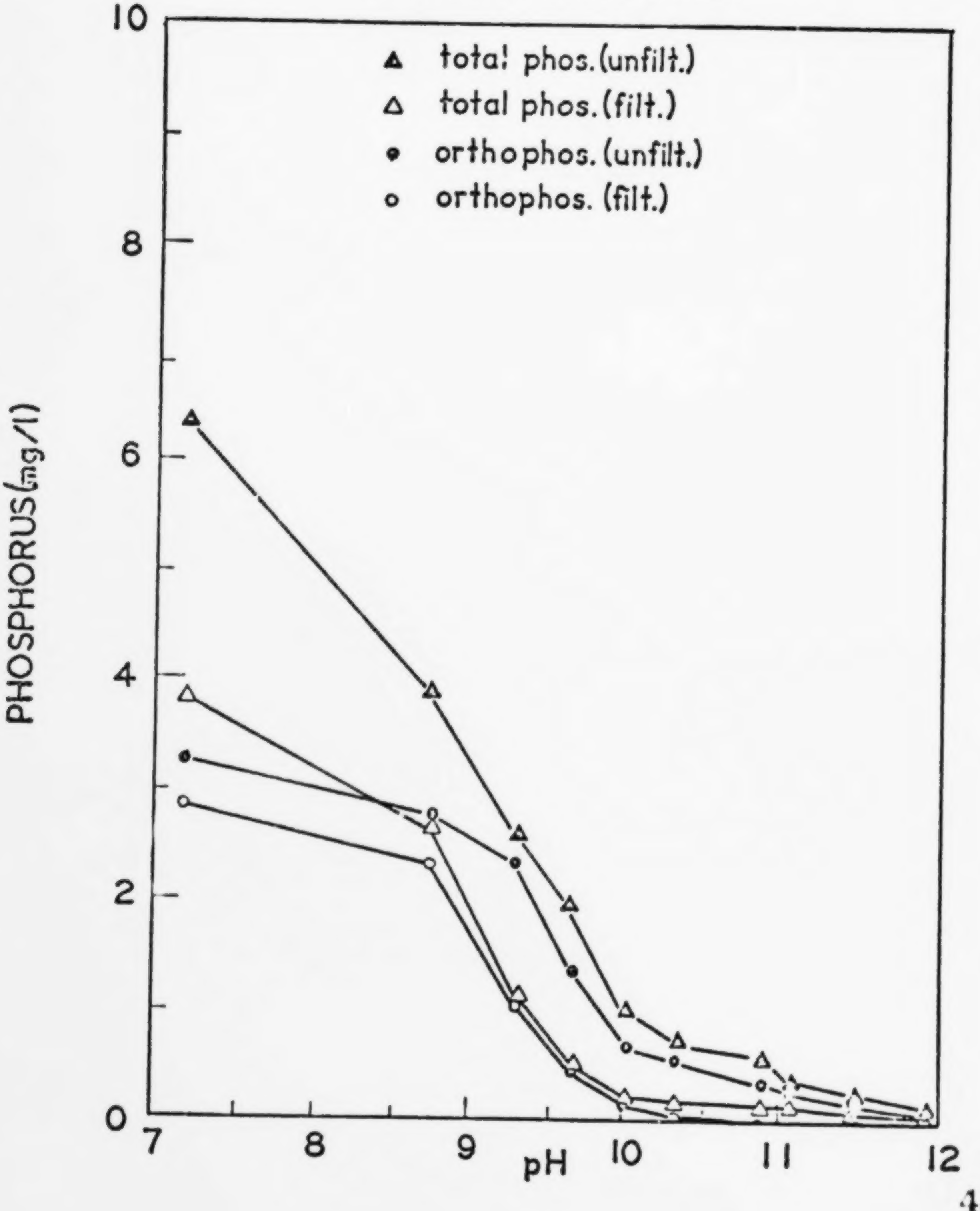
FIGURE 11

LIME PRECIPITATION AND PHOSPHORUS REMOVAL FROM RAW WASTEWATER.



and two-stage lime clarification are shown in Fig. 12. In the two-stage process 350-400 mg/l as CaO was added to obtain a pH of 11.8-12.0. For the single-stage process 250 mg/l of lime was added to a pH of 9.5. Both processes achieved a residual P content after media filtration of 0.4 and <1 mg/l P, respectively, or 96 and > 89 percent removal. Thus, while high alkalinity waters require more lime to reach any desired pH, phosphorus removal can be obtained with lower pH's.

Thus far, I have emphasized the phosphorus removal capabilities of chemical precipitation and have only mentioned that clarification is also obtained concurrently. The data in Fig. 12 illustrates that with good phosphorus removal, equally good clarification is obtained. In the two-stage process, lime clarification of primary effluent obtained 67.6 percent removal of COD and 75 percent removal following filtration. Similarly, the single-stage process obtained 65 percent COD removal after lime precipitation and filtration. These excellent removals of COD by precipitation and filtration compare well with secondary treatment of wastewater by trickling filters.

4. Sludge Handling

One of the more important factors to be considered for choice of chemical to use is the sludge handling characteristics. This subject will be covered in more detail elsewhere. Iron sludges and particularly alum sludges from a clarifier are likely to contain only about 0.5 percent solids and will dewater slowly. Hence costs for concentrating and disposal may be high. In contrast, lime sludge, particularly sludge containing calcium carbonate from high alkalinity waters is granular and dewaters easily. Lime sludge from the Lebanon operation, for example, is removed as a 3 percent slurry from the clarifier, then thickened in a gravity settler to a 15 percent slurry which is pumped to sand drying beds. The sludge dewaters on the beds to 50 percent solids. Dewatering can also be obtained on vacuum filters or by centrifuges. In contrast to alum or iron for which no practical method for recovery of metal is available, lime can be recalcined to recover useful calcium oxide. Work at Lake Tahoe demonstrated that recalcined lime recovered from a tertiary lime treatment is as effective as virgin lime in the treatment process.

5. Cost of Treatment

Cost of chemical precipitation is subject to so many variables as choice of chemical, point of addition, and waste characteristics, etc., as pointed out earlier, that this discussion can do no more than indicate approximate costs. Fig. 13 illustrates the total cost of single-stage lime clarification for plant sizes ranging from 1 to 250 mgd. The cost is shown to range from 6.09¢/1000 gal for the smallest plant to 3.78¢/1000 for the largest. Savings are indicated when lime is recalcined.

Figure 12 Results of Treatment by Lime Clarification, Filtration and Carbon Adsorption of Primary Effluent at Washington, D. C. — (I) and Lebanon, Ohio (II)

	Two-Stage Lime Clarification - Low Alkaline Wastewater(I)		Single-Stage Clarification - High Alkaline Wastewater(II)	
	mg/l	% Removal	mg/l	% Removal
Primary Effluent				
Phosphorus, P	10.4		8.8	
TOC	78.4		76	
BOD	139		76	
COD	265		192	
Lime Clarified Effluent				
Phosphorus, P	0.5	94.8		
TOC	27.1	65.2		
BOD	42.0	69.7		
COD	86	67.6		
Filtered Effluent				
Phosphorus	0.39	96.2	< 1	> 89
TOC	22.6	69.9	26	52.3
BOD	28	80.0	25	67.2
COD	66	75.0	67	65.0
Carbon Effluent				
Phosphorus	-		< 1	> 89
TOC	6.5	90.6	10	87
BOD	4	97	10	87
COD	11	95.8	27	86

Figure 13 Total Cost of Phosphate Removal for Single-Stage Lime Clarification

(Cents per 1,000 gallons)

	Size of Plant			
	1 mgd	10 mgd	100 mgd	250 mgd
Capital amortization	1.25	1.12	.84	.77
Land amortization	.12	.12	.12	.12
Operating and maintenance	2.30	.79	.50	.47
Cost of chemicals				
Lime	1.75	1.75	1.75	1.75
Cost of sludge disposal by hauling to land fill (25-mile one-way trip)	.67	.67	.67	.67
TOTAL	6.09	4.45	3.88	3.78
Savings if lime can be reclaimed	-.67	-.67	-.67	-.67
TOTAL (with recalcining)	5.42	3.78	3.21	3.11

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ADVANCED WASTE TREATMENT PLANTS FOR TREATMENT OF SMALL WASTE FLOWS

I. J. Kugelman, W. A. Schwartz, and J. M. Cohen

Introduction

During the last two decades, the need for waste treatment in localities where connection to a centralized system is not possible or economically feasible has become more and more urgent. Typical examples include suburban housing developments, institutions, rest areas along highways, tourist areas in parks, isolated work camps, etc. In these situations the flow is low because the population served is limited, and consequently such plants are referred to as "small flow" treatment plants. As a rule of thumb, plants in excess of 500,000 gal/day capacity (equivalent population 5,000) are considered outside the "small flow" class. This size limit is based on limitations of shop fabrication and typical sizes of communities not connected to regional sewerage systems, but is somewhat arbitrary. Data on the number of plants of this type which will be required in the near future are virtually non-existent. Table 1 presents some estimates of the present "market" for such plants. An overall estimate of the significance of this problem is given by the data of Smith which show that 10% of the total weight of BOD in municipal sewage discharges is generated in communities with a population below 5,000.

Performance Criteria

The "small flow" treatment plant market has not been ignored by commercial interests. During the last 20 years they have installed approximately 5,000 package sewage treatment plants. These have all been designed on the basis of biological treatment technology and for the most part they have done the job required. However, these plants are limited in applicability. In many locations, which shall or already do require "small flow" systems, one or more of the following conditions can be expected which will exclude the use of a purely biological unit:

a. Toxic materials in the waste
b. Extreme diurnal flow and load variation
c. Requirement for nutrient removal
d. Requirement for start and stop operation
e. Reuse of the treated effluent

A pertinent example of the inability of biological package plants to meet recent upgrading in treatment criteria is given by the recent action of several localities halting subdivision home construction pending the installation of adequate waste treatment. Biological package plants did not meet the stringent effluent criteria established by these local governments.

TABLE 1

ESTIMATES - SMALL FLOW TREATMENT PLANT NEEDS

CATEGORY	ESTIMATED NO. OF PLANTS
SMALL COMMUNITIES	3,000 - 6,000(a)
PARKS AND MARINAS	2,000 - 10,000(b)
ALASKAN WORK CAMPS & ESKIMO VILLAGES	500 - 2,000

(a) The Economics of Clean Water - Vol. I, USDI, March 1970.

(b) Personal Communication - U.S.Park Service

TABLE 2

TYPICAL "SMALL FLOW" WASTE CHARACTERISTICS

	Typical(a) Municipal Sewage	Household(b) Wastewater	Houseboat(c) Wastewater	Winter(d) Recreation Area
SUSPENDED SOLIDS, mg/l	200	376	173	342
COD, mg/l	350	776	460	790
BOD, mg/l	210	435	222	390
TOTAL NITROGEN, mg/l--N	33	84	67	78
AMMONIA NITROGEN, mg/l--N	12	64	--	23
TOTAL PHOSPHATE, mg/l PO_4	24	61	49	32
GREASE, mg/l	14	68	92	---

(a) Water Supply & Sewerage, Steel, E. W., McGraw Hill - 1960

(b) Watson, et al., JWPCF, 39, 2039 (Dec 1967)

(c) Clark, B. D., "Houseboat Waste Characteristic & Treatment," PR-6,FWQA (Sept.1967)

(d) Clark, B.D., "Basic Waste Characteristics at Water Recreational Areas, PR-7, FWQA (Aug. 1968).

Waste treatment technology which can meet the stringent performance criteria for small flow plants has been under development in the Advanced Waste Treatment program for the last decade. These new methods are primarily physical and chemical in nature rather than biological. Demonstration of these concepts on a large scale (> 10 mgd) is soon to take place. As these systems have worked well in pilot plants, it is reasonable to assume they shall do so in "small flow" systems. These pilot plant studies were conducted on typical domestic sewage. Although small flow wastes are in general stronger than typical domestic sewage they are not fundamentally different in character. Table 2 compares the characteristics of typical small flow wastes and typical domestic sewage.

Small Flow Advanced Waste Treatment Plants

A. Basic design consideration

Treatment of small flow wastes may require one or more of the following steps: suspended solids removal, dissolved organic carbon removal, nitrogen and/or phosphorus removal, and disinfection. The Advanced Waste Treatment program has developed one or more processes to meet each of these treatment requirements. However, design of a treatment train cannot be predicated only on achieving a desired effluent quality but must be concerned with reliability and cost. On these latter points conditions are somewhat different when comparing small volume and conventional flow plants. Because of the highly variable nature of the wastes to be treated a premium must be placed on processes which function well regardless of waste variability. A second factor to consider is the apportionment of total cost between capital and operation. In most "small flow" situations a full-time operator is not economically feasible, thus a treatment system with a relatively high capital expenditure for automation can be justified.

Based on the special requirements of small flow treatment plants discussed above, three generalized treatment trains have been selected as worthy of consideration for future development.

B. Physical-Chemical System

Figure 1 illustrates a wholly physical-chemical treatment system employing chemical clarification followed by carbon adsorption. Either powdered or granular activated carbon can be utilized. The system performance is quite flexible in that the degree of treatment obtained can be varied by the use of optional filtration steps and by altering dosage of chemicals or powdered carbon or the frequency of replenishment of granular carbon. In some cases activated carbon treatment may not

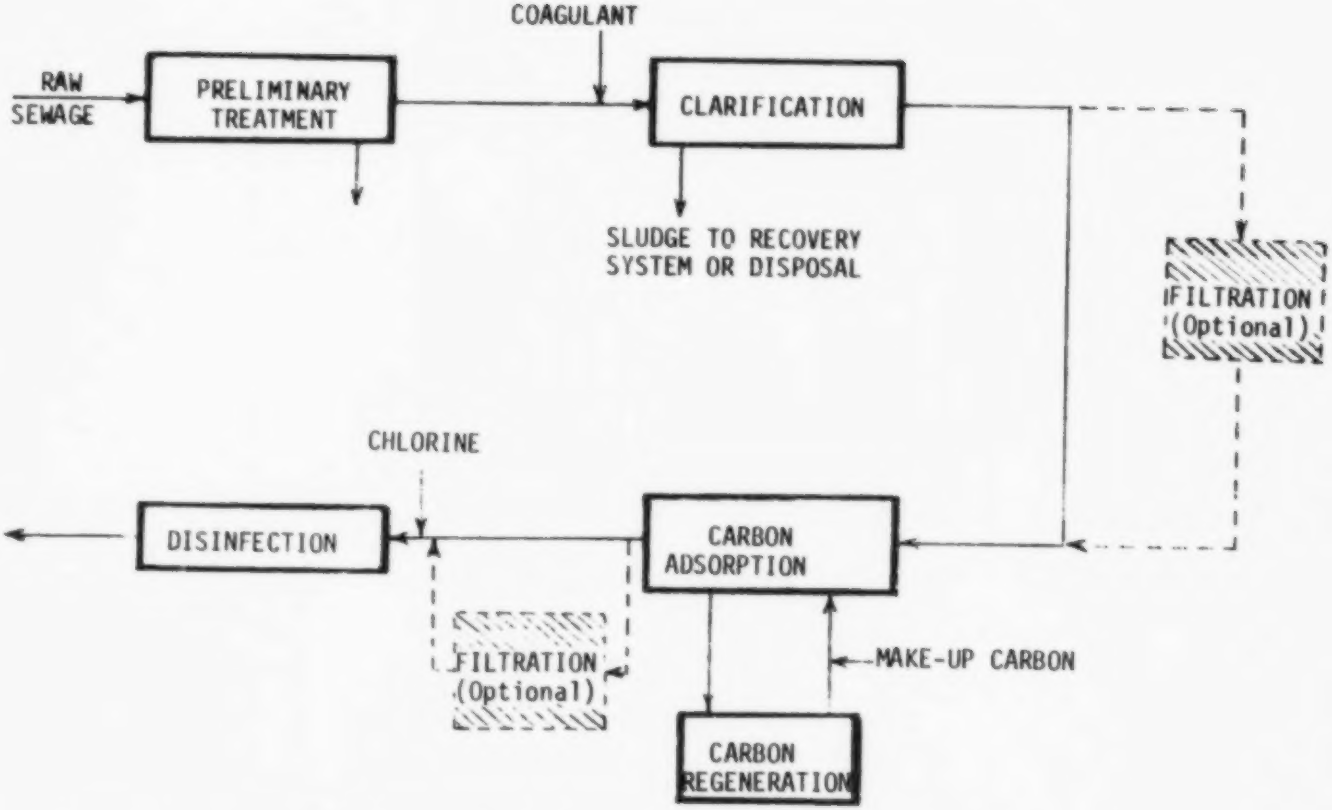
RAW
SEWAGE
PRELIMINARY TREATMENT
COAGULANT
CLARIFICATION
SLUDGE TO RECOVERY SYSTEM OR DISPOSAL
FILTRATION (Optional)
CHLORINE
DISINFECTION
CARBON ADSORPTION
FILTRATION (Optional)
MAKE-UP CARBON
CARBON REGENERATION

Fig. 1. SCHEMATIC FLOW DIAGRAM OF A PHYSICAL-CHEMICAL TREATMENT SYSTEM

be required as chemical clarification consequently yields 70% to 75% COD removal from raw sewage. It is anticipated that only for the largest scale small flow plants will carbon regeneration be provided. For most of these plants the cost of discarding carbon after one use appears less than the required capital expenditure for a regeneration system.

Chemical clarification with a sufficient dose of iron or aluminum salts or lime can provide any degree of phosphorus removal required. In general, the extra chemical cost associated with dosages to the level required for phosphorus removal is relatively minor in the overall cost figures for small volume treatment systems. In addition, dosage to this level will always assure excellent clarification.

Ammonia nitrogen removal is not provided for specifically in this scheme. The most feasible method for nitrogen removal would be breakpoint chlorination, because it is the method which requires least attention from an operator. Thus effluent disinfection and ammonia nitrogen removal can be achieved in one step. However, a significant increase in chloride ion can be expected in the effluent.

C. Ultrafiltration System

The second general treatment scheme is illustrated in Figure 2. This provides for a variety of optional pretreatment steps leading to an ultrafiltration membrane assembly. A membrane providing total suspended solids capture with some degree of dissolved organic removal would be utilized. In essence, the role of the membrane in these systems is to function as a polishing or safety device which will produce a consistent effluent quality regardless of fluctuations in the raw waste. The pretreatment steps serve to reduce the load on the membrane and reduce the rate of membrane fouling.

D. Chemical-Biological System

Figure 3 illustrates a chemical-biological treatment system. This treatment train is less generally applicable than the others in that it is heavily oriented to biological treatment. It is only applicable where neither toxicity nor flow interruption is anticipated. In this system mineral addition to the aerator is included not only for phosphorus removal but to prevent biological solids loss from the sedimentation tank under varying hydraulic load. Provision of an automatic backwash deep bed filter serves as an effluent polishing device. Nitrogen removal may be obtained with this system by operation of the activated sludge system such that nitrification will take place and injecting sufficient methanol ahead of the filter so that denitrification will take place.

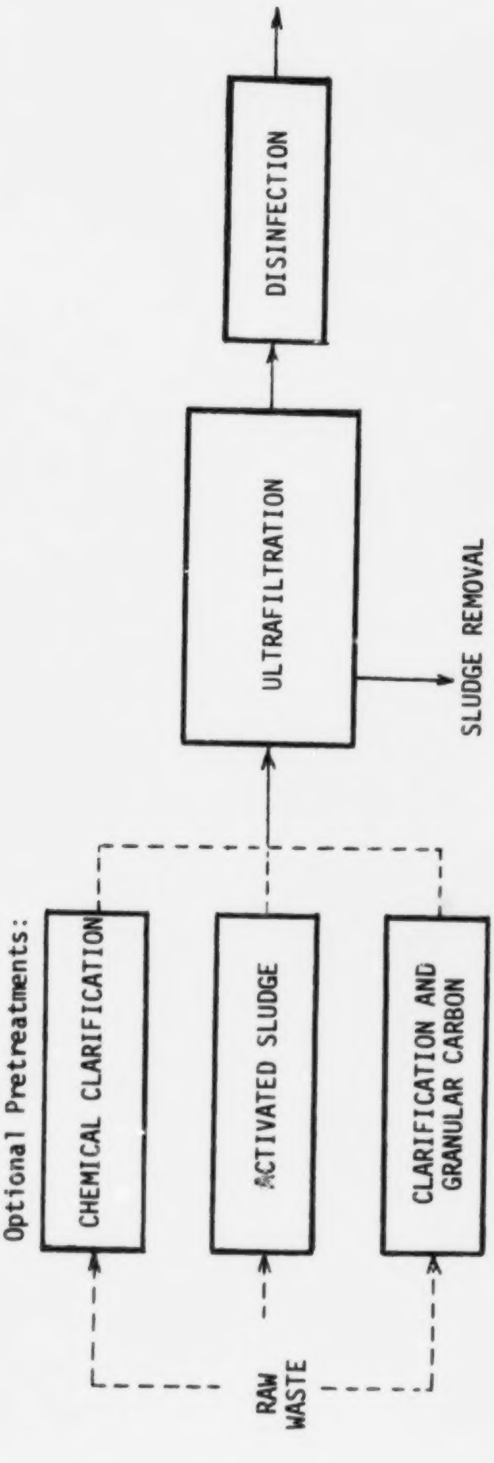
Optional Pretreatments:
CHEMICAL CLARIFICATION
ACTIVATED SLUDGE
CLARIFICATION AND GRANULAR CARBON
RAW WASTE
ULTRAFILTRATION
SLUDGE REMOVAL
DISINFECTION

FIGURE 2. SCHEMATIC DIAGRAM OF ULTRAFILTRATION SYSTEM

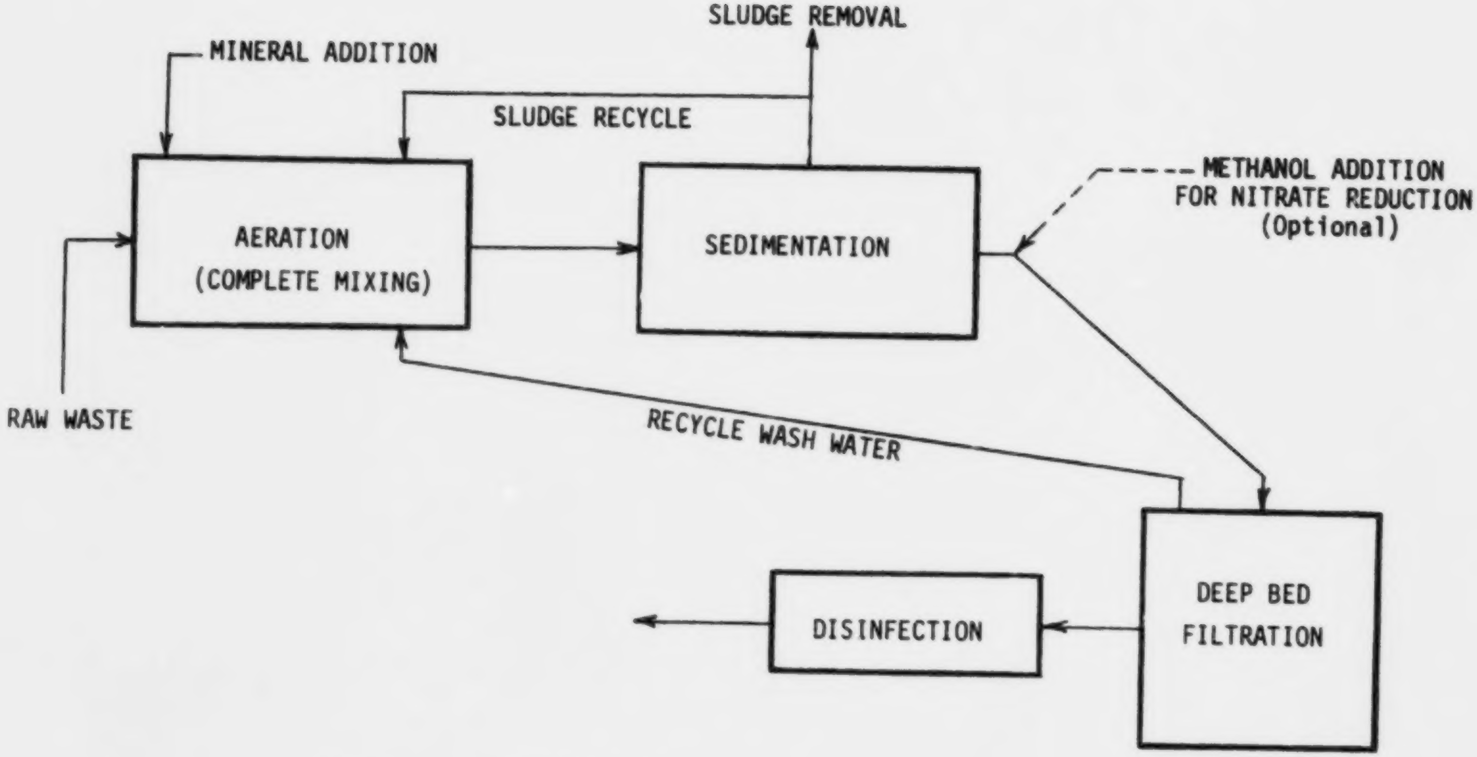
MINERAL ADDITION
SLUDGE REMOVAL
SLUDGE RECYCLE
METHANOL ADDITION FOR NITRATE REDUCTION (Optional)
AERATION (COMPLETE MIXING)
SEDIMENTATION
RAW WASTE
RECYCLE WASH WATER
DEEP BED FILTRATION
DISINFECTION

FIGURE 3. SCHEMATIC DIAGRAM OF CHEMICAL-BIOLOGICAL SYSTEM

Sludge Treatment

As is the case for large scale plants the greatest problem for small flow plants is sludge disposal. Of the several sludge disposal methods available some such as sludge drying beds, are more applicable to small plants than large plants, while some such as mechanical sludge filtration appear to be too expensive for small plants. Treatment and disposal methods which are being seriously considered include:

a. dewatering on drying beds with or without lime treatment
b. gravity thickening and temporary storage with periodic pump-out
c. incineration
d. aerobic digestion

Incineration is the most costly of these but the most attractive as the only residue for disposal is a sterile ash. A fluidized bed incineration system which has been successfully applied to the incineration of plastic wastes will soon be evaluated for incineration of sludge. This system shows promise for small treatment plants because fluidized beds hold their temperature well during shutdown of several days, and are relatively easy to automate.

Our knowledge of fundamental characteristics of the chemical-biological or chemical raw sewage solids sludges which will result from small flow systems is not as extensive as our knowledge of conventional primary and secondary sludges. These data will be required in order to design sludge handling systems. In future research activities in the small flow treatment plant field, gathering data on sludge handling will be of prime importance.

Current Models of Small Advanced Waste Treatment Plants

There are very few examples of small Advanced Waste Treatment plants which are commercially available at the present time. However, these are worth discussing in some detail in order to demonstrate their inherent advantages. In principle, physical-chemical plants differ basically from biological plants in that they can be designed to be much more compact than the latter, and so lend themselves better to small or modular installations. Moreover, since all such Advanced Waste Treatment plants consist of 2-3 distinctly different physical-chemical treatment processes in series, obviously these varied elements can be assembled in a variety of ways to serve any specific desired purpose and to minimize space requirements. An array of processes is available to effect: solids removal, organic removal, nutrient removal, and, where necessary, some degree of inorganic removal. Since even biological processes require solids removal, it is possible (as we shall see) to substitute an Advanced Waste Treatment process for the traditional gravity settling to effect an improvement in a biological package plant, both in terms of treatment efficiency and space requirements.

There are several small plants with which we have some experience. The small Advanced Waste Treatment plants which we shall consider are of interest for three basic reasons: 1) they represent several distinctly different process sequences; 2) they represent some of the combinations of the physical-chemical treatment processes which are most likely to be applied in the future, both at large or small scale; and 3) they demonstrate applications in situations where small Advanced Waste Treatment plants hold great promise for the future. Four different plants shall be considered, three of which have been tested at a commercially applicable scale, and the other of which is still being tested in the laboratory.

Clarification-Carbon Treatment

The most popular physical-chemical treatment design currently being evaluated consists of chemical clarification followed by granular activated carbon adsorption, with or without an intermediate filtration step. (See Figure 1). The Met-Pro Water Treatment Company of Lansdale, Pennsylvania*, has designed a unit of this type (Model 1100-10) for the treatment of laundry wastes. However, the plant has wider applicability.

The Met-Pro unit (see Plate 1) consists of an upflow solids contact clarifier followed by two-stage granular carbon treatment: the first stage is a downflow column of carbon on an anthracite coal base and the second stage is an upflow carbon column. The unit is rated at about 24,000 GPD. In our evaluation of this unit at Cincinnati, a comminuted and degritted wastewater was dosed with any of several coagulants just prior to entry into the sludge blanket, where the retention time was about one hour. Supernatant overflowed a weir onto a 33" deep bed of carbon (12 x 40 mesh) at a rate of between 1.3 and 2.5 gpm/sf. The second (upflow) stage had the same amount of carbon as the first, and the empty bed contact time for both stages was 16-32 minutes for our range of flows. Carbon effluent was used to backwash the upflow carbon bed. The operation is countercurrent in that spent carbon from the upflow bed is used to replace spent carbon from the preceding downflow bed. Spent carbon from the first contactor is discarded.

Before proceeding to the results of the plant evaluation, we should consider the application intended for the plant so that the data can be put in proper perspective. The situation is typical of many for which this and similar small plants are envisioned, i.e., a remote area where no other waste treatment facilities are within reach and where some degree of water reuse is projected because water supply is either in short supply or expensive.

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*Mention of commercial products does not imply endorsement by the Federal Water Quality Administration, U. S. Department of the Interior.

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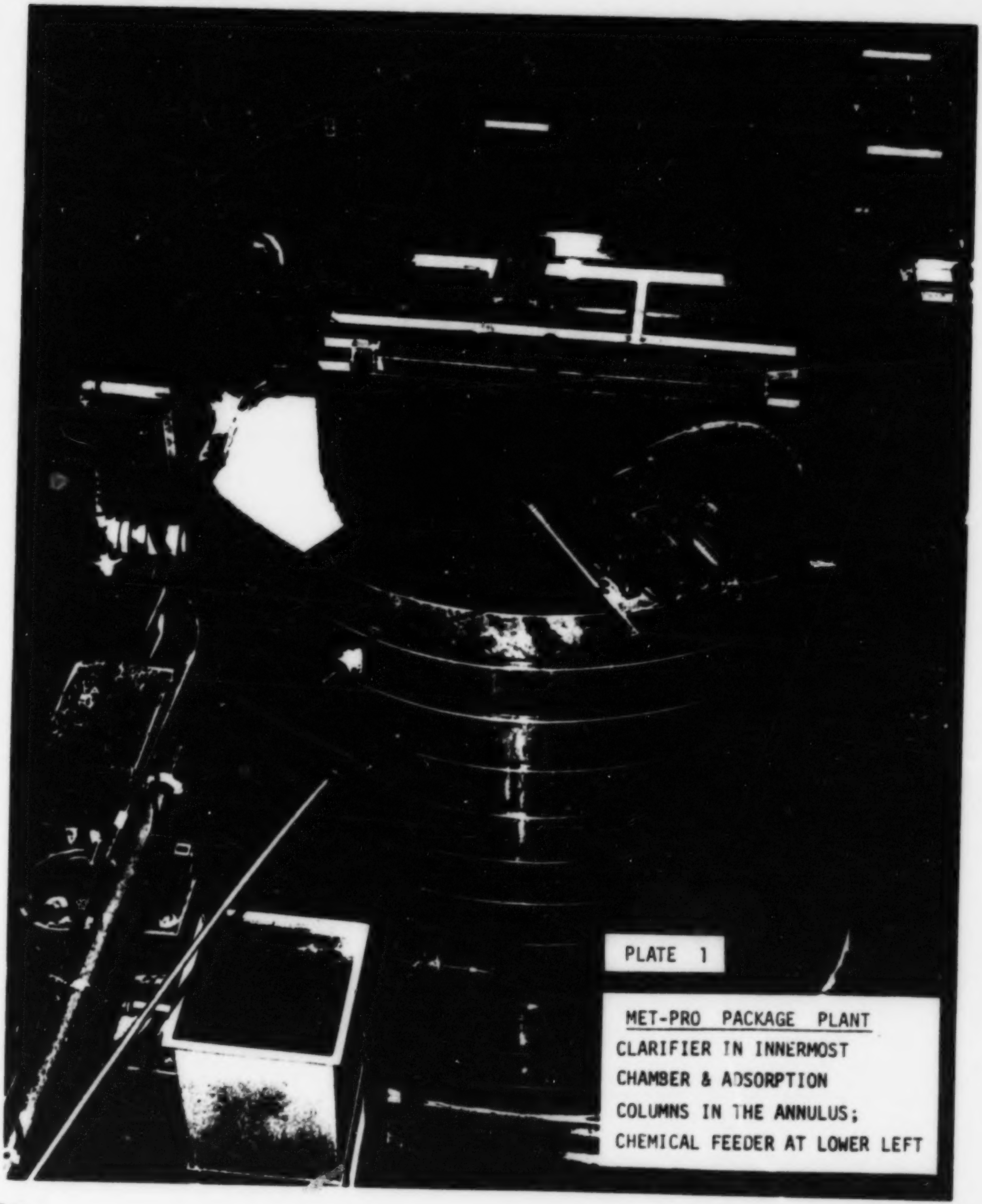

PLATE 1

MET-PRO PACKAGE PLANT

CLARIFIER IN INNERMOST CHAMBER & ADSORPTION COLUMNS IN THE ANNULUS; CHEMICAL FEEDER AT LOWER LEFT

The North Slope of Alaska is a developing area of oil production. Being remote and highly susceptible to the effects of pollution, a treatment-reuse scheme is a desirable goal. The Met-Pro plant was thought suitable to serve the needs of work camps (see Plate 2) and two such plants have been installed for waste treatment (see Plate 3). The total water use of a 300-man camp might be 30,000 GPD. The principal opportunity for reuse is for toilet-flushing, which would not require drinking water quality. This use represents a very significant fraction of the total water use, perhaps 6,000-8,000 GPD.

The Met-Pro plant was operated during 3-4 months on an 8-hour-per-day basis. This permitted us to evaluate the effect of starting and stopping. About 200,000 gallons were treated as part of 26 separate runs which tested various coagulants, dosages, flow rates, and raw wastewaters. A summary of the results is shown in Table 3. Results for all the runs were averaged. The organic effluent level is obviously much superior to biological secondary effluent. Microbiological quality is still somewhat poor and disinfection might be in order. Some idea of the variability of influent and effluent characteristics can be gained from the graphs showing frequency distributions for COD, turbidity, and phosphate (Figures 4-6). It should be noted that both COD and turbidity are always at least of secondary effluent quality. The sharply inclined line for PO_4 effluent is obviously meaningless here because high PO_4 values are due to inadequate coagulant dosages.

The chemical costs are given in Table 4. The costs are FOB and do not include freight.

Poliovirus I inactivation by disinfection was evaluated and the results are given in Table 5. Two separate trials were made using both iodine and chloramine. It is important to notice that even where water is being used for non-potable purposes, the water must be thought of as potentially for human contact. Therefore, aesthetic and microbiological criteria are just as much of a concern as solids and organic removal standards.

Activated Sludge-Ultrafiltration Treatment

A quite different sort of small Advanced Waste Treatment plant is represented by the biological "package" plant which has been upgraded by addition of an Advanced Waste Treatment process. The IOPOR System is manufactured by Dorr-Oliver, Inc., of Stamford, Connecticut*. It consists of a high solids activated sludge process followed by an ultrafiltration membrane for complete solids separation. The use of the membrane to replace settling or conventional filtration is intended to conserve space and to achieve a

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PLATE 2

WORK CAMP IN ALASKA

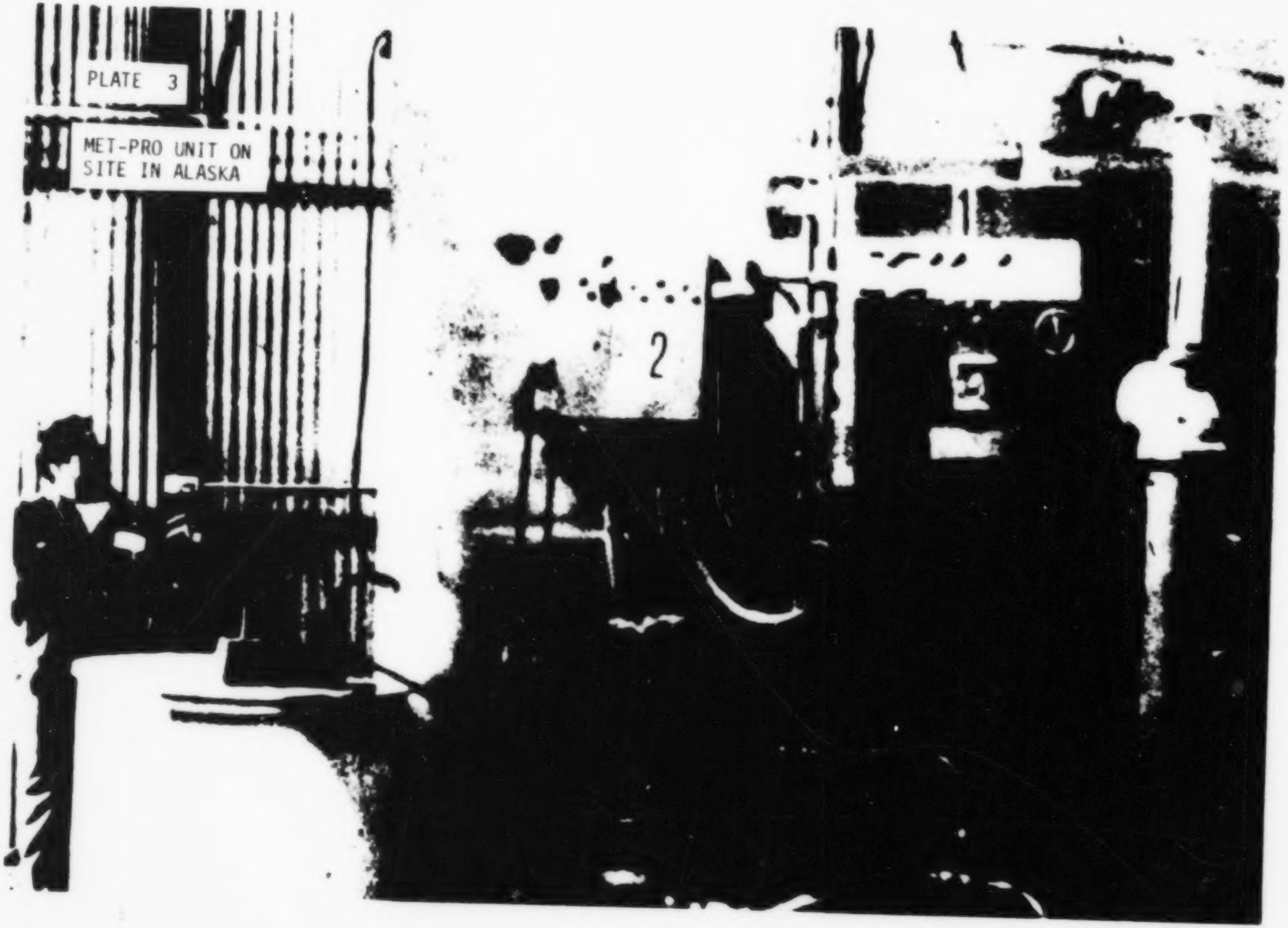

PLATE 3

MET-PRO UNIT ON
SITE IN ALASKA

TABLE 3
SUMMARY OF PERFORMANCE
OF THE MET-PRO CLARIFICATION-CARBON PACKAGE PLANT
(FWQA, CINCINNATI, OHIO)

PARAMETER	AVERAGE VALUES, mg/l INFLUENT	EFFLUENT	% REMOVAL
COD	330	17	95
COLOR (UNITS)	48	8.3	83
TURBIDITY (JTU)	91	2.5	97
pH	7.6	--	--
TSS	151	--	--
TOTAL P	6.7	< 1.1	--
AMMONIA-N	21.1	13.3	--
ORGANIC-N	7.8	1.6	--
TOTAL COUNT (PER ML)	6.45×10^6	170,800	97.4
TOTAL COLIFORM (PER ML)	278,000	1,418	99.5
FECAL COLIFORM (PER ML)	59,000	175	99.7
FECAL STREPTOCOCCI (PER ML)	13,840	91	99.3

FIGURE 4

DISTRIBUTION OF INFLUENT & EFFLUENT COD (MET-PRO PACKAGE PLANT)

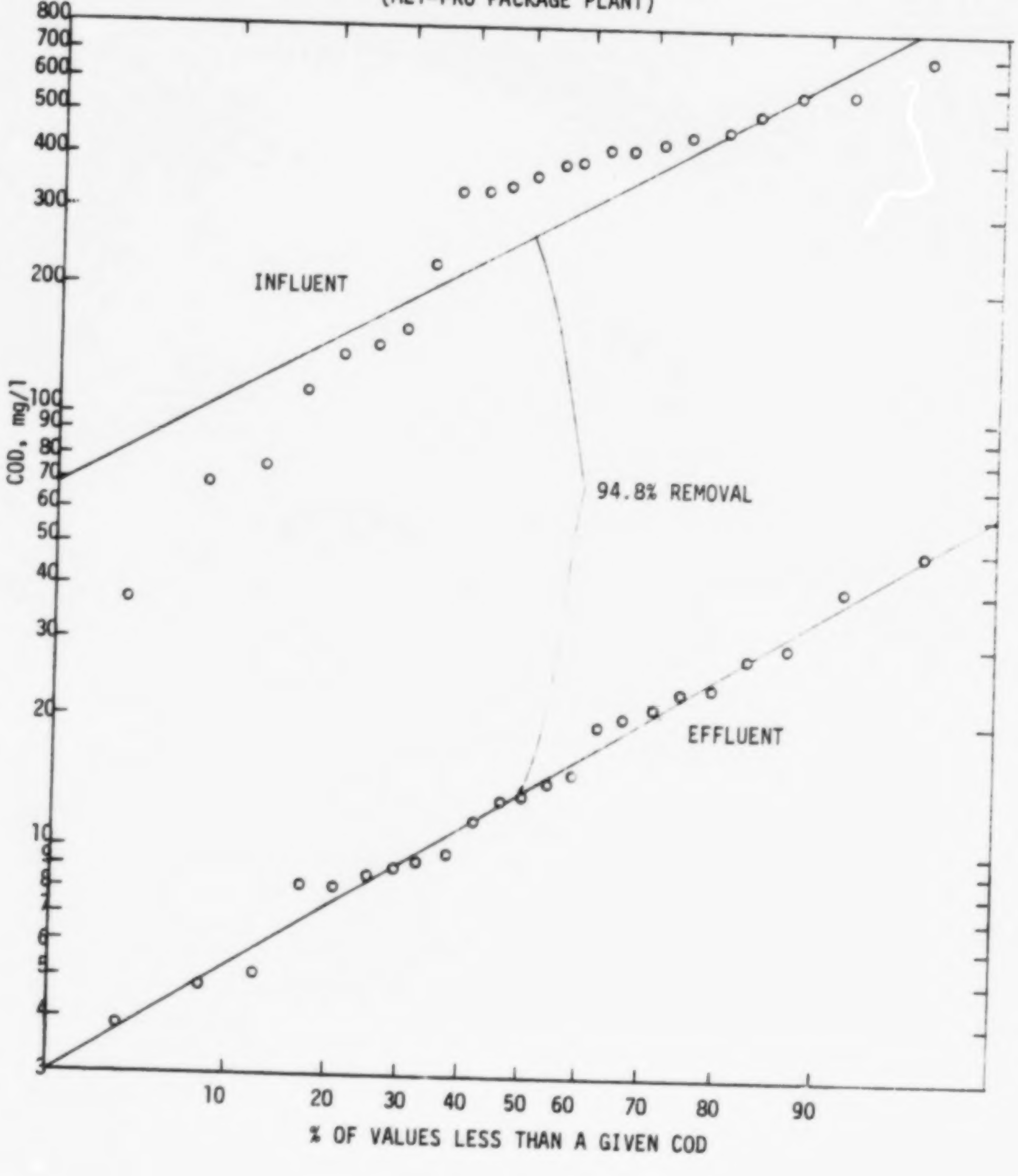

FIGURE 5
DISTRIBUTION OF INFLUENT & EFFLUENT TURBIDITY
(MET-PRO PACKAGE PLANT)

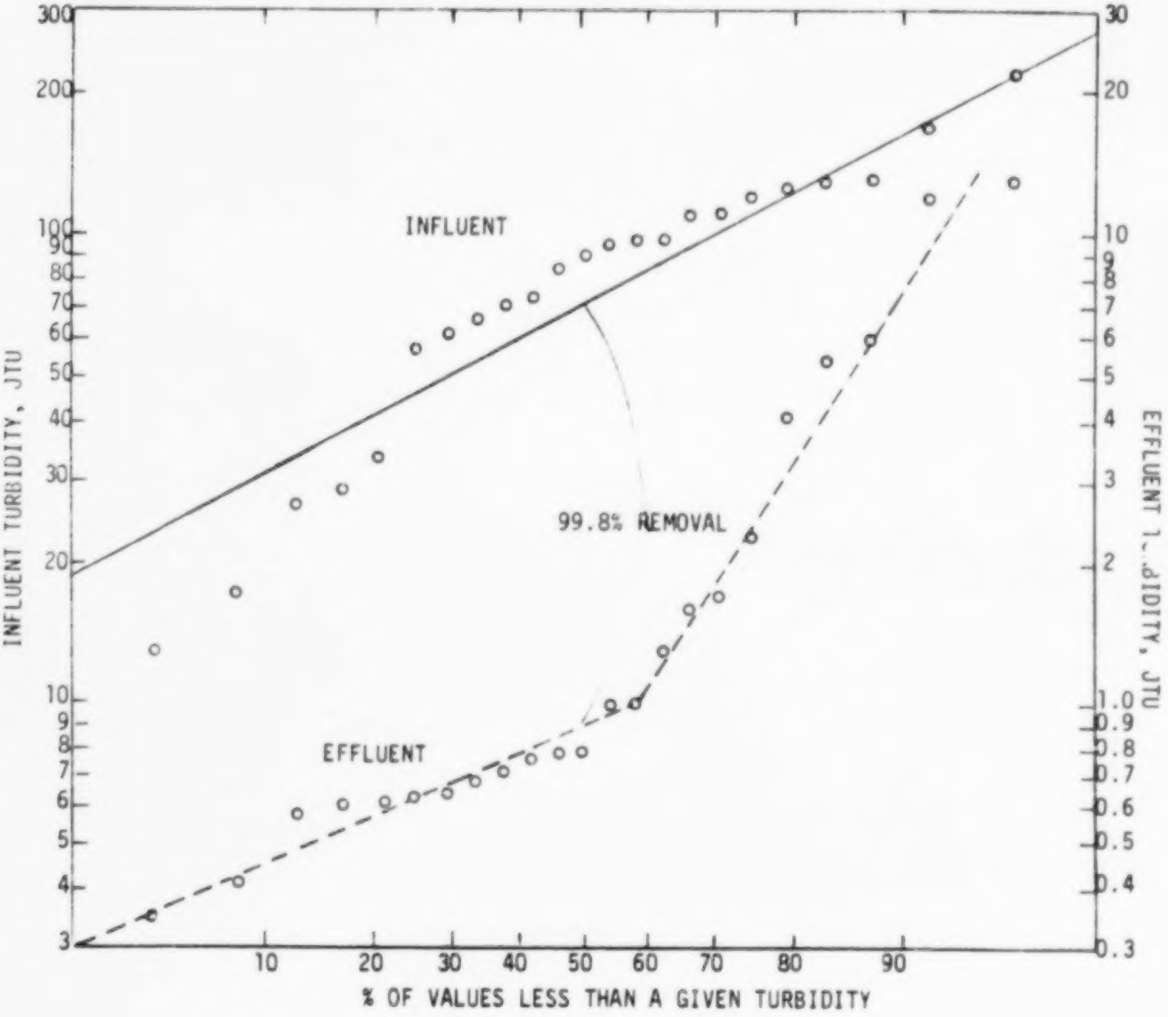

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FIGURE 6

DISTRIBUTION OF INFLUENT & EFFLUENT PO_4-P
(MET-PRO PACKAGE PLANT)

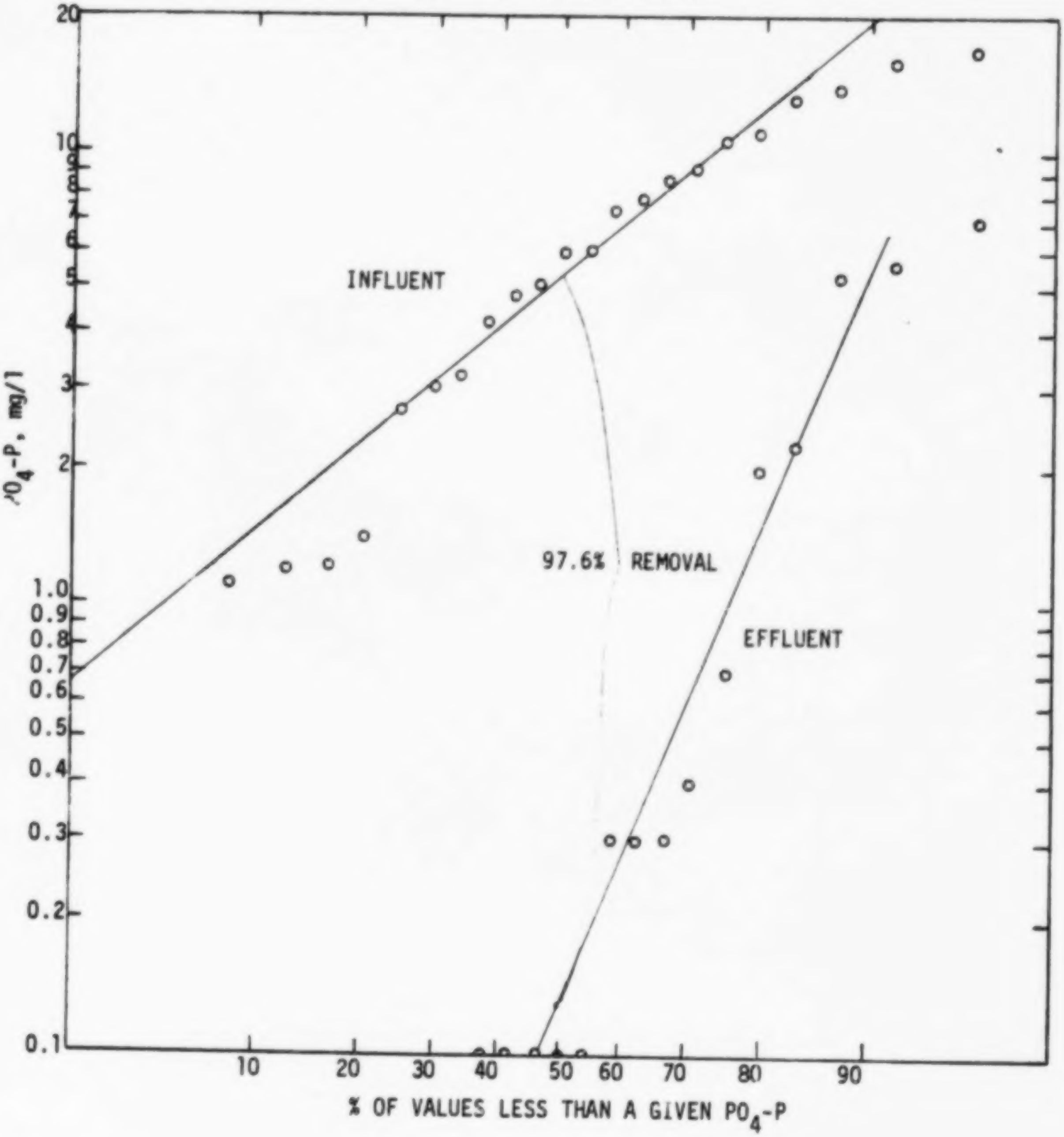

TABLE 4

CHEMICAL COSTS
FOR THE MET-PRO PACKAGE PLANT
(FWQA, CINCINNATI, OHIO)

	QUANTITY (1000 gal)	COST (per lb)	COST (per 1000 gal)
ALUMINUM SULFATE	2.51 lb	2.91¢	7.3¢
FERRIC SULFATE	2.51 lb	2.16¢	5.4¢
ACTIVATED CARBON			
*capacity = 0.05	1.67 lb	32¢	53.5¢
capacity = 0.75	1.25 lb	32¢	40.0¢
capacity - 1.00	0 84 lb	32¢	26.8¢

LOW: FERRIC SULFATE AND CARBON (cap. = 1.00) = 32.2¢

HIGH: ALUMINUM SULFATE AND CARBON (cap. = 0.50) = 60.8¢

*capacity of the carbon to remove organics in: lbs COD/lb carbon

TABLE 5

POLIOVIRUS I INACTIVATION AT 15°C
IN THE MET-PRO PACKAGE PLANT
(FWQA, CINCINNATI, OHIO)

RUN NO.	DISINFECTANT	CONCENTRATION mg/1		pH	KILL TIME (minutes)	
		DOSE	RESIDUAL		99%	99.9%
I	IODINE	5	3.0	7.3	16	25
	IODINE	1	0.2	7.5	84	150
	CHLORAMINE	15	14.9	7.3	25	60
II	IODINE	5	3.7	7.5	11	20
	IODINE	2	1.6	7.4	27	44
	CHLORAMINE	22	17.7	7.6	6	24

higher solids removal, the latter being necessary in view of the high solids mode of activated sludge operation which is used.

Ultrafiltration is the lesser known counterpart of hyperfiltration, or reverse osmosis, a process of increasing interest for desalination applications. Ultrafiltration by definition removes only solids and no inorganic salts. Although it requires a sophisticated hydrodynamic system similar to that of RO, it is operated under a much lower pressure, usually about 50 psi versus the 400-1500 psi required for RO. Like RO, it has a problem with membrane flux decline, but this is usually controlled by routine hydrodynamic cleaning techniques. Since ultrafiltration is presently more expensive than settling or filtration, its principal current application is in situations such as small Advanced Waste Treatment plants where space is important, and where the product quality and consistency of supply is vital.

The IOPOR System to be discussed is currently located on the top of Pikes Peak (14,110 ft) in Colorado, and is thus perhaps the world's highest sewage treatment plant. As with the case of Alaska, Pikes Peak is a remote site where pollution hazards in the water-shed are great and where water supply is likewise a difficult matter. (See Plate 5). The wastes are generated largely by the tourist facilities on the Peak. (See Figure 7).

Formerly, large quantities of potable water had to be trucked up the mountainside over tortuous roads from the 10,000 foot level. This was used for both potable and non-potable purposes. However, since the vast majority of this demand was actually for non-potable water, specifically for toilet-flushing, reuse of treated wastewater for this purpose will make the water supply task considerably less difficult. The recent loss of a tank truck over the mountainside emphasizes the hazards of water supply to the Peak. Toilet-flushing represents a high proportion of the total use because other uses are minimal. Little food preparation is done on the premises. The IOPOR unit is designed to treat 15,000 GPD, leaving only roughly 5,000 GPD to be trucked up the mountain for drinking and washing water.

Since the population and tourist trade of Pikes Peak is projected to increase steadily, both the water supply and pollution control aspects are obviously of high priority. Moreover, this demonstration at a remote site is potentially applicable for many other similar sites such as resorts and parks. Special plant design requirements here include: high product quality, both analytically and aesthetically, and ability to function for extended periods on recycled flow (from overnight to several days, up to 100% recycle).

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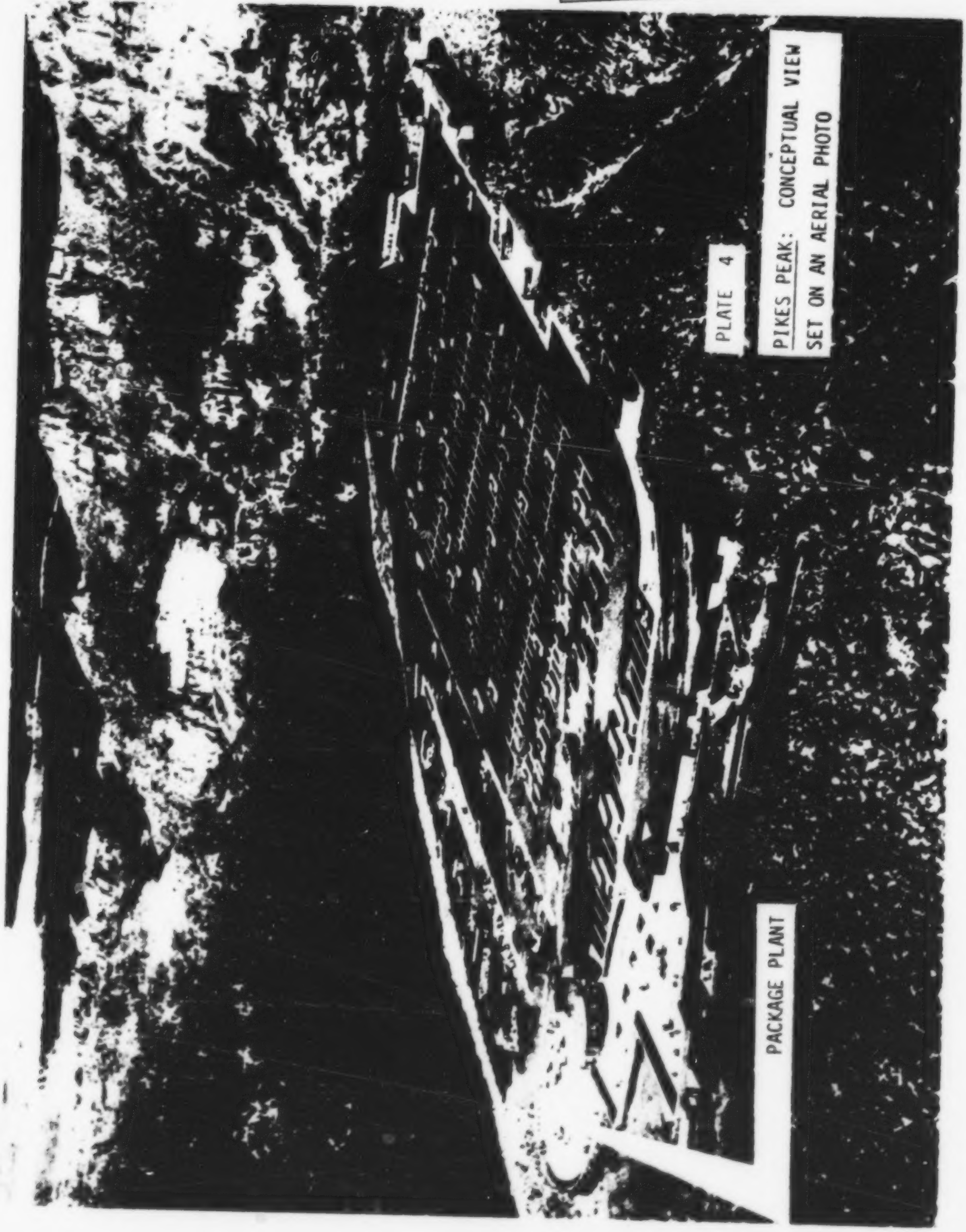

PLATE 4

PIKES PEAK: CONCEPTUAL VIEW SET ON AN AERIAL PHOTO

PACKAGE PLANT

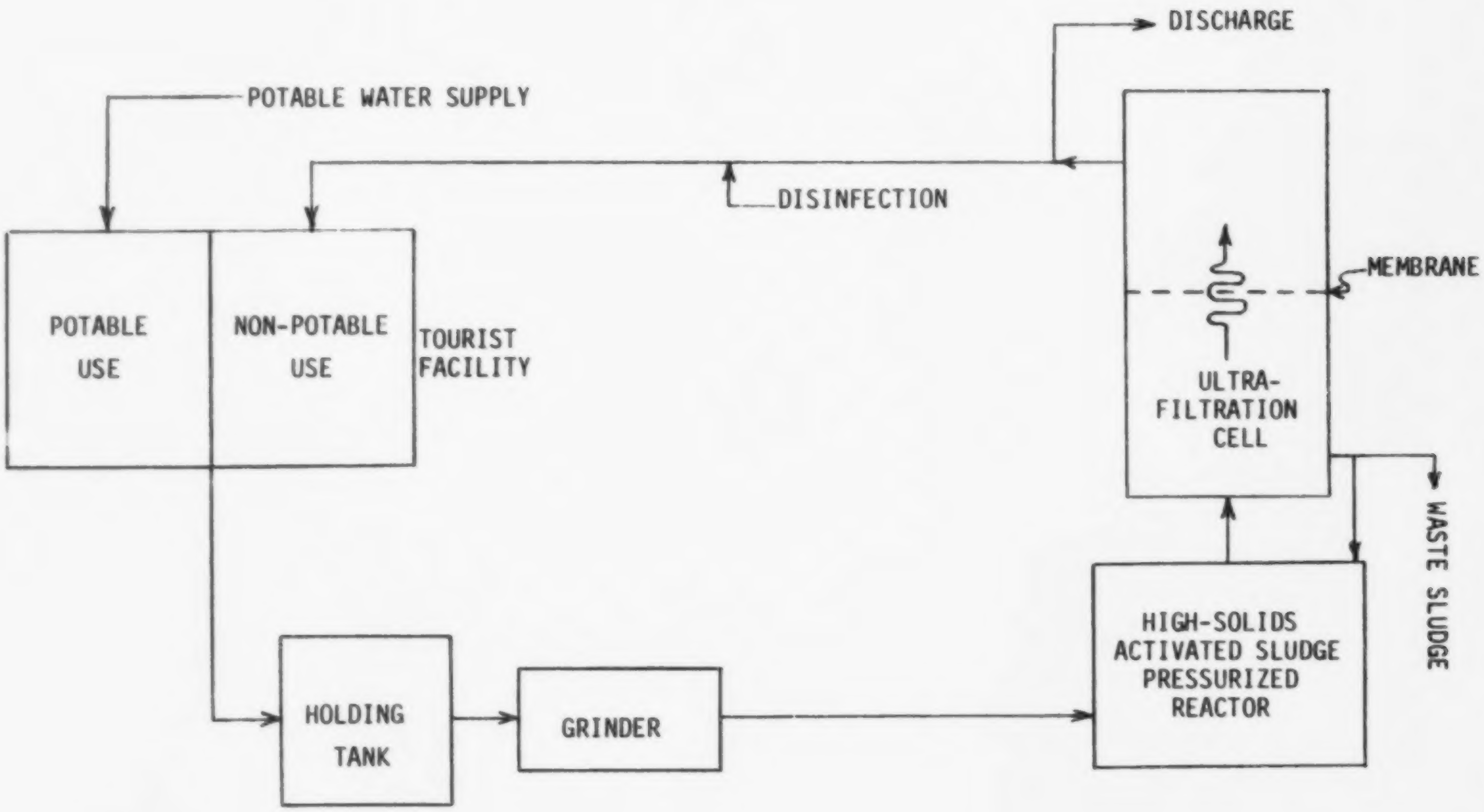
DISCHARGE
POTABLE WATER SUPPLY
DISINFECTION
MEMBRANE
POTABLE USE
NON-POTABLE USE
TOURIST FACILITY
ULTRA-FILTRATION CELL
WASTE SLUDGE
HIGH-SOLIDS ACTIVATED SLUDGE PRESSURIZED REACTOR
HOLDING TANK
GRINDER

Fig. 7. SCHEMATIC FLOW DIAGRAM OF THE PIKES PEAK TREATMENT & REUSE SYSTEM

The IOPOR unit was tested for several months at Colorado Springs, during which time mechanical difficulties were corrected. (See Plate 5). A summary of results for the preliminary testing are given in Table 6. Raw sewage from the city was used as a feed. The unit performed extremely and consistently well under a very heavy loading in every respect except perhaps color. Effluent organics were generally of tertiary quality and even the odor was minimal. Frequency distributions for influent and effluent COD are shown in Figure 8; effluent COD was always better than secondary quality.

The IOPOR unit was moved to Pikes Peak in August 1970 as soon as necessary site construction was completed. Membrane modules and plastic parts were removed with the onset of winter in late September, when the Peak facilities and roads were closed. In the meantime, about 3 weeks of operation was completed which included some recycling experience but no direct reuse. Operation with full reuse will commence next summer. A summary of results for operation on the Peak is given in Table 7.

Operation was generally good as in the preliminary trial, even though the waste was somewhat different. The Peak waste has a higher proportion of human wastes than normal municipal waste. Color was again on the high side, but not necessarily a problem. It may be that a small amount of powdered carbon will improve the color. Phosphorus removal was typical of biological treatment. Physical-chemical treatment plants would be superior in this respect if phosphorus were of concern here. However, it is possible that phosphorus removal, if desired, could be improved by adding a precipitant to the aerator. COD and turbidity frequency distributions are shown in Figures 9 and 10. Not much variation is evident for either of them, illustrating that both consistent waste strength and consistent plant performance can probably be expected.

The membrane flux decline is shown in Figure 11. The initial flux of 30 gfd was the result of washing the membrane after the preliminary trial. Although flux declined to 7-8 gfd in 20 days, no washing was done so the flux decline shown is misleading. If, however, it continued at the rate shown, it would be illustrated by the extrapolated dashed line. The flux would be about 2.5 gfd at 6 months and 1.5 gfd at 1 year. Six months coincidently represents both the membrane guarantee period and the approximate upper limit for the length of any one tourist season. Washing the membrane daily or several times weekly would keep the flux above the design flow (15,000 GPD = 7.8 gfd) for six months.

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PLATE 5

DORR-OLIVER PACKAGE PLANT:
ACTIVATED SLUDGE AT CENTER,
FLANKED BY MEMBRANE BANKS,
CONTROLS IN FOREGROUND

TABLE 6

SUMMARY OF PERFORMANCE OF THE DORR-OLIVER ACTIVATED SLUDGE-ULTRAFILTRATION PLANT PRELIMINARY RESULTS FROM COLORADO SPRINGS PRIOR TO OPERATION AT PIKES PEAK, MARCH-JUNE, 1970

PARAMETER	INFLUENT mg/1	EFFLUENT mg/1	% REMOVAL
BOD	382	< 1	> 99
COD	678	20	97
TOC	192	7.5	96
TURBIDITY (JTU)	-	< 0.1	-
COLOR (UNITS)	-	28	-
TSS	323	0	100
MLSS	5510	-	-
COLIFORM (PER 100 ML)	-	0	100
PO_4-P	12.2	7.7	37
pH	7.02	6.62	-
THRESHOLD ODOR NUMBER	-	5.6	-
FLUX	10.1 GFD = 19,400 GPD		

FIGURE 8

DISTRIBUTION OF INFLUENT & EFFLUENT COD
FROM THE DORR-OLIVER AS-UF PLANT
(PRELIMINARY TRIAL)

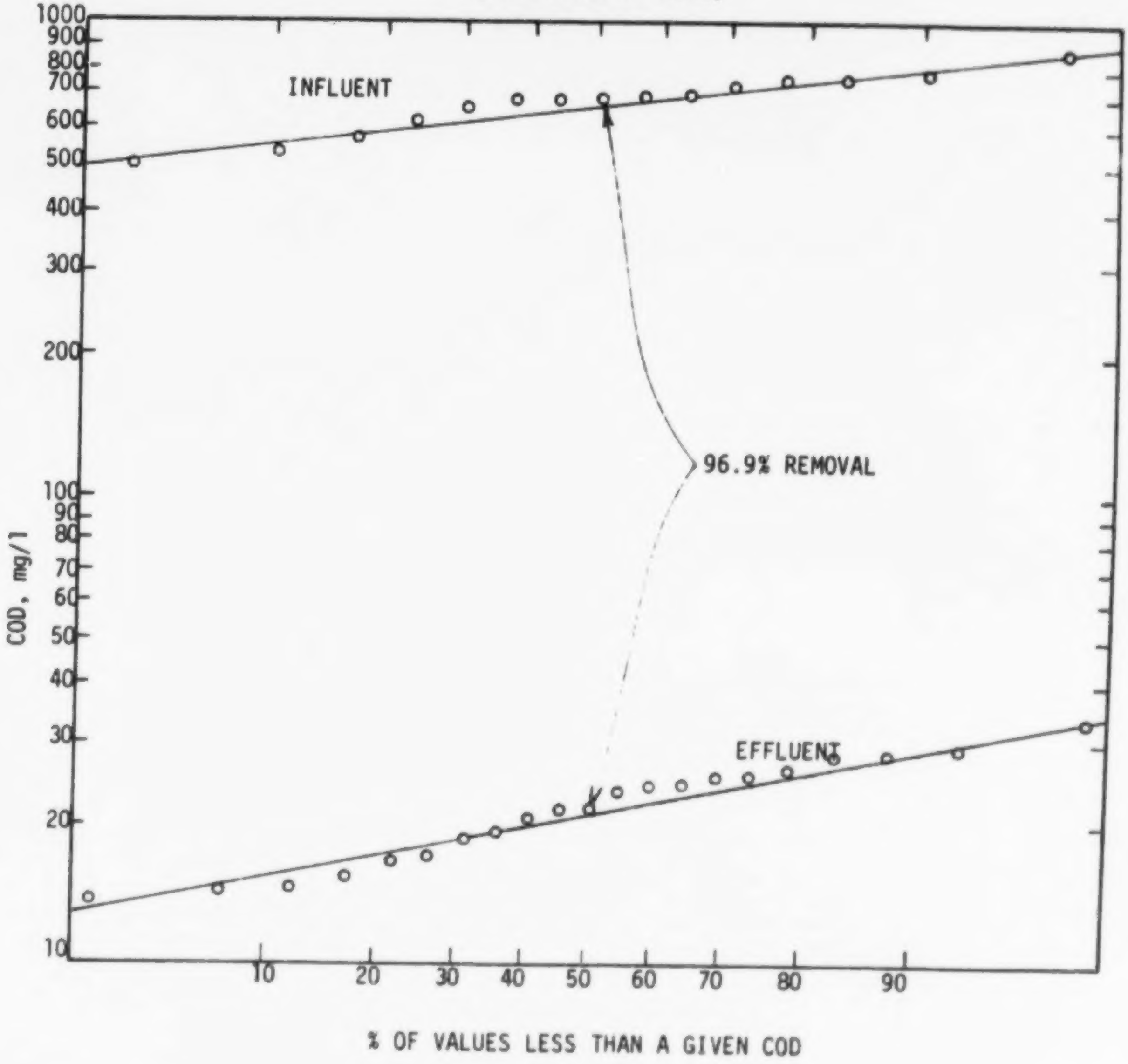


TABLE 7

SUMMARY OF PERFORMANCE
OF THE DORR-OLIVER ACTIVATED SLUDGE-ULTRAFILTRATION PLANT
OPERATIONS AT PIKES PEAK
AUGUST-SEPTEMBER, 1970

PARAMETER	INFLUENT mg/l	EFFLUENT mg/l	% REMOVAL
BOD	285	<1	>99
COD	547	32	94
TOC	136	6.6	95
TURBIDITY (JTU)	47	0.33	-
COLOR (UNITS)	320	40	-
TSS	129	0	100
MLSS	3954	-	-
COLIFORM (PER 100 ML)	-	0	100
PO_4-P	9.1	11.1	-
pH	7.9	5.9	-
THRESHOLD ODOR NUMBER	-	6	-
AVERAGE FLUX	11.0 GFD = 21,000 GPD		

FIGURE 9

DISTRIBUTION OF INFLUENT & EFFLUENT COD
FROM THE DORR-OLIVER AS-UF PLANT
(PIKES PEAK)

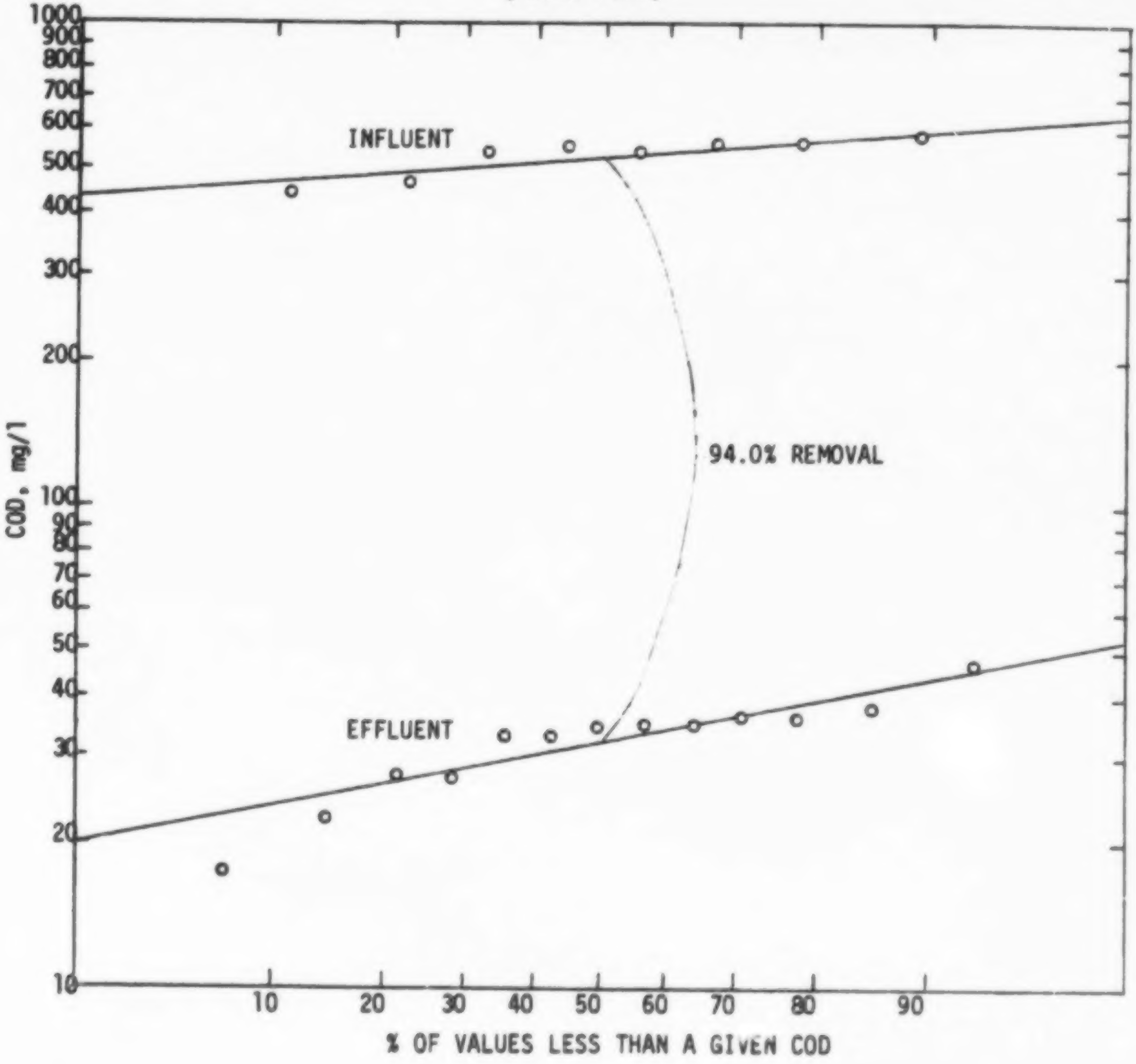
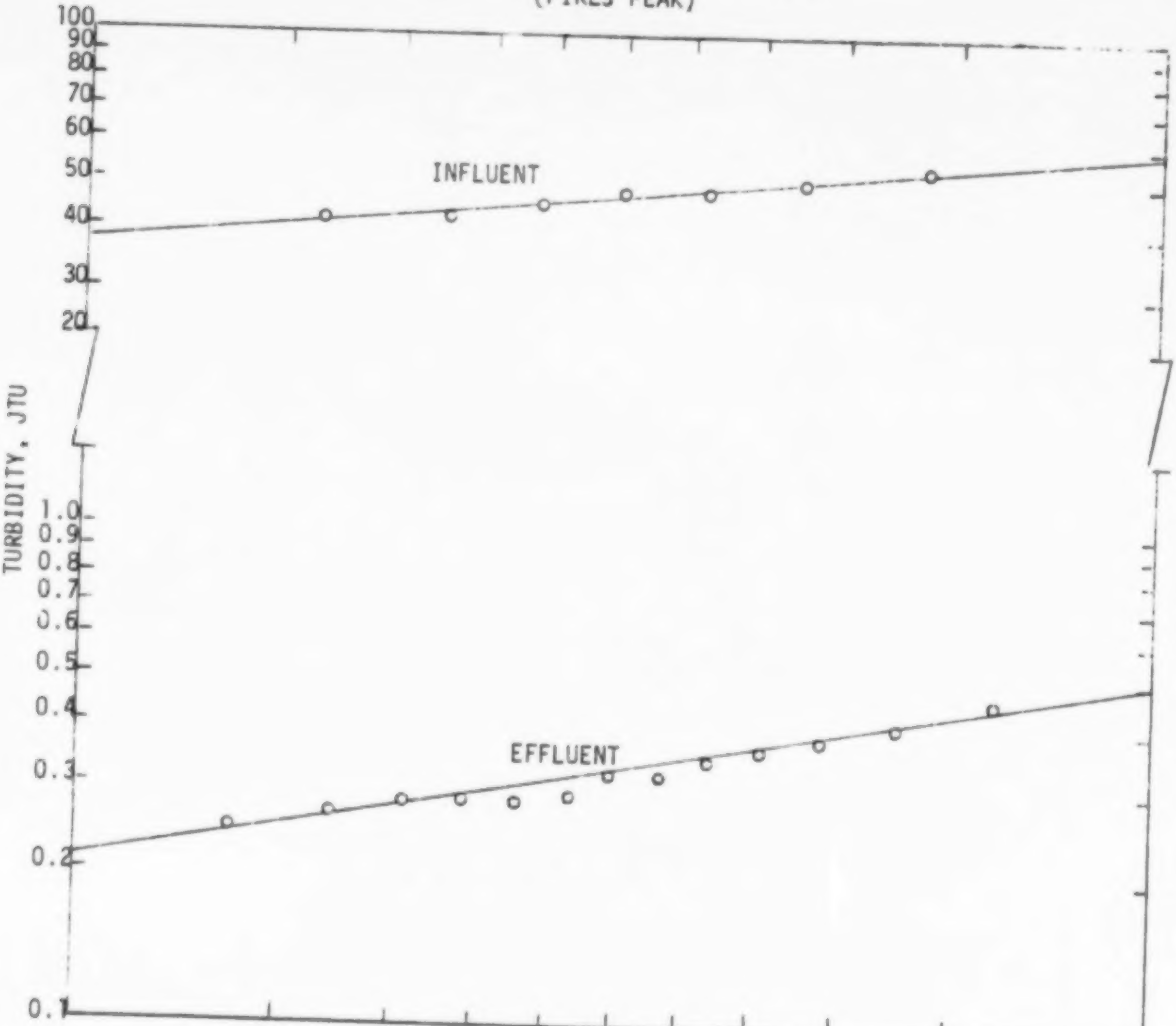


FIGURE 10

DISTRIBUTION OF INFLUENT & EFFLUENT TURBIDITY
FROM THE DORR-OLIVER AS-UF PLANT
(PIKES PEAK)



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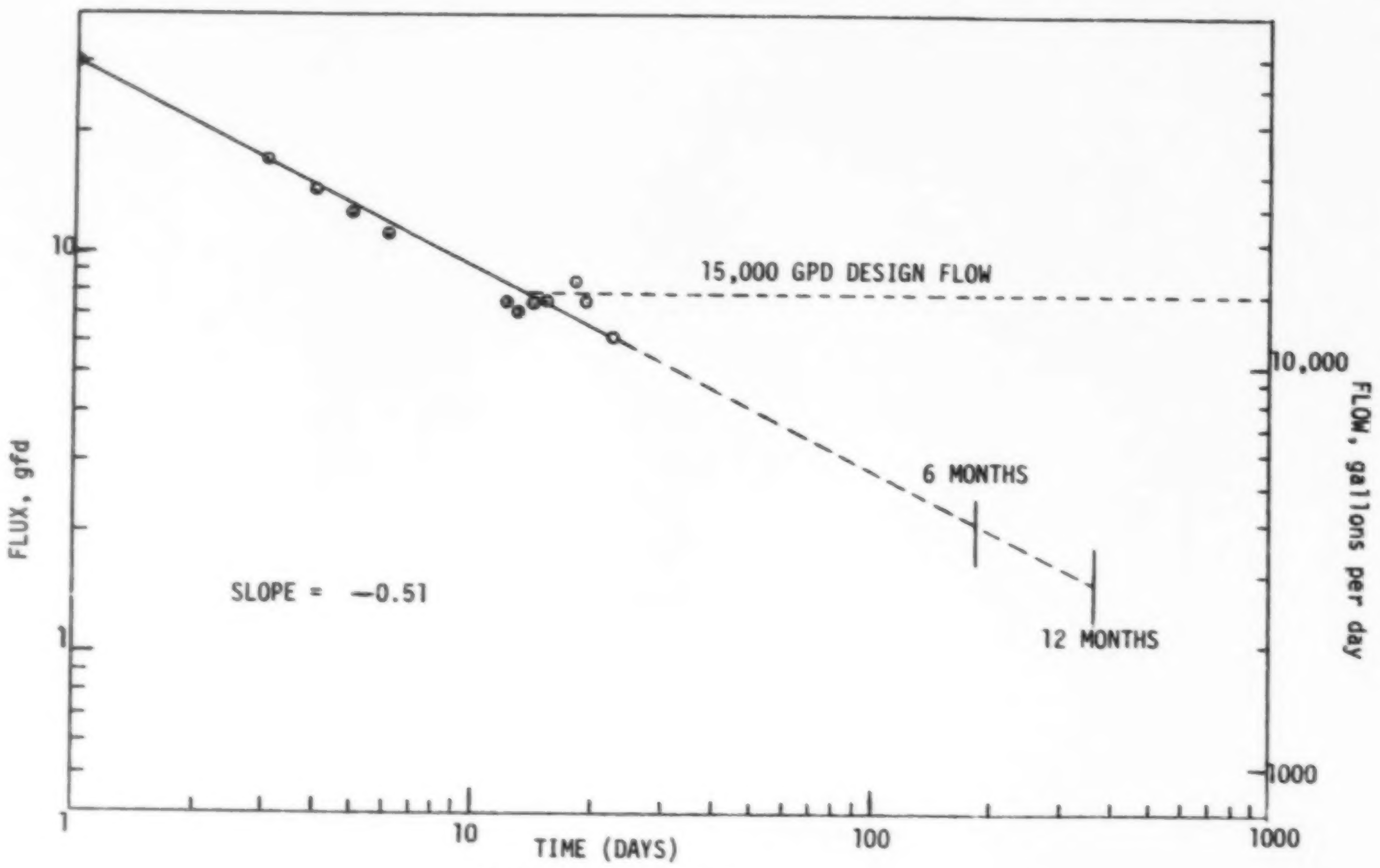

MEMBRANE FLUX DECLINE
IN THE DORR-OLIVER AS-UF PLANT
(PIKES PEAK)

FIGURE 11

Clarification by Moving Bed Filtration

An interesting concept which has applicability to treatment of small flows is the moving bed filter developed by Johns-Manville Corporation Manville, New Jersey*. This device operates at hydraulic loading rates normally associated with deep bed rapid filters but at solids loadings usually applied only to clarifiers. It provides clarification superior to that achieved by a clarifier but in a space similar to that required by a rapid filter. Figure 12 illustrates the operation of this device. Raw waste is dosed with appropriate chemicals and flows into a tank which provides head for the filter operation. The sewage filters downward through the inclined packed bed of sand to a screened pipe and thence flows to a collector. Sewage solids and floc collect primarily on the filter face although some depth filtration is obtained. When the head loss exerted by the accumulated solids becomes excessive the sand bed is pushed upward and a cutter slices off the top layers of sand and suspended solids. The sand-sludge mixture is collected in the bottom of the head tank and is then pumped to a sand washer. Clean sand is returned to a hopper and eventually to the bottom of the sand bed.

This system has been tested with raw sewage, primary effluent, and trickling filter effluent at the Bernards Township Sewage Treatment Plant. The pilot plant had a capacity of 10 gpm at the average hydraulic loading 2 gpm/sf. Alum and an anionic polyelectrolyte were used as the treatment chemicals. The results obtained in two weeks operation on raw sewage are given in Table 8. Excellent phosphorus, suspended solids, and BOD removals were obtained. In fact, the MBF produced treatment superior to that achieved by the Bernards plant during this period (which consists of secondary treatment by trickling filter).

This work shall be expanded to other coagulants, and to evaluate the performance when powdered activated carbon is added to the feed to obtain additional organic removal. It is expected that the cost of once-used carbon required to insure organic carbon removal to the level of secondary treatment will be economically feasible for small flow application.

Johns-Manville estimates that the total cost of a 1 MGD plant is 12.0¢ per 1000 gallons. This estimate includes all operation, maintenance, chemicals (alum and polymer) and amortization of the capital expenditure of $264,000.

Clarification-Carbon-Ultrafiltration Treatment

A potential small Advanced Waste Treatment plant is presently being evaluated in the laboratory and with a small prototype model by Oak Ridge National Laboratory. No significant amount of data has been

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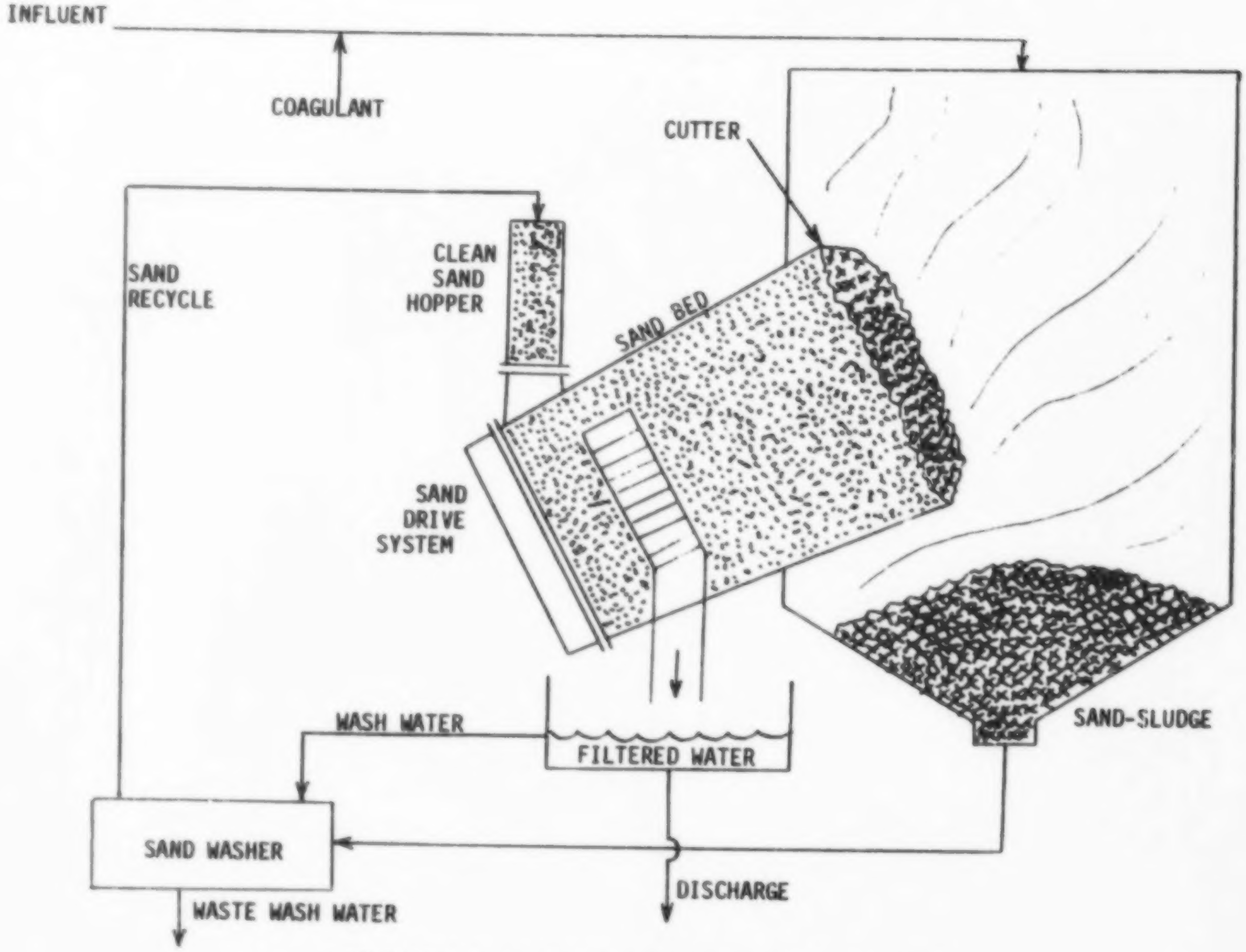
*Mention of commercial products does not imply endorsement by the Federal Water Quality Administration, U. S. Department of the Interior.


INFLUENT
COAGULANT
CUTTER
CLEAN SAND HOPPER
SAND RECYCLE
SAND BED
SAND DRIVE SYSTEM
SAND-SLUDGE
WASH WATER
FILTERED WATER
SAND WASHER
DISCHARGE
WASTE WASH WATER

SCHEMATIC FLOW DIAGRAM OF THE MOVING BED FILTER
(AFTER JOHNS-MANVILLE)
FIGURE 12

TABLE 8

SUMMARY OF PERFORMANCE OF THE MOVING BED FILTER IN THE TREATMENT OF RAW SEWAGE (AFTER JOHNS-MANVILLE)

PARAMETER	RAW SEWAGE	MBF EFFLUENT	% REMOVAL
	(Average Values, mg/1)		
PHOSPHORUS			
TOTAL	21.5	2.16	90
FILTERABLE	18.6	0.79	96
ORTHO-	13.2	0.57	96
pH	7.2	7.0	--
TSS	156	27	83
TURBIDITY (JTU)	119	16	87
BOD	115	19	84

NOTE: COAGULATED WITH 200 mg/1 ALUM AND ≈0.5 mg/1 MAGNIFLOC 860A collected yet, but the device is nonetheless worth mentioning.

This plant scheme consists of chemical clarification, powdered carbon contacting, and ultrafiltration separation of solids, as shown in Figure 13. The wastewater is contacted with coagulant as usual (so far, iron salts have been used, but lime is probably equally applicable) and flocculated, but no separation of floc is effected yet. A small dose of powdered activated carbon is added. The floc-carbon suspension is then pumped into the ultrafiltration cell where solids are separated. So far in lab tests, considerable fractions of organics as well as all of the solids are removed from the water with a membrane flux of about 100 gfd, or almost 10 times the IOPOR flux.

The IOPOR membrane used in the Dorr-Oliver plant is typical of conventional membranes. It is cast from a synthetic polymer on a rigid support in the form of parallel plates. Casting conditions as well as casting solution ingredients are closely controlled to assure the desired flux and rejection for the membrane.

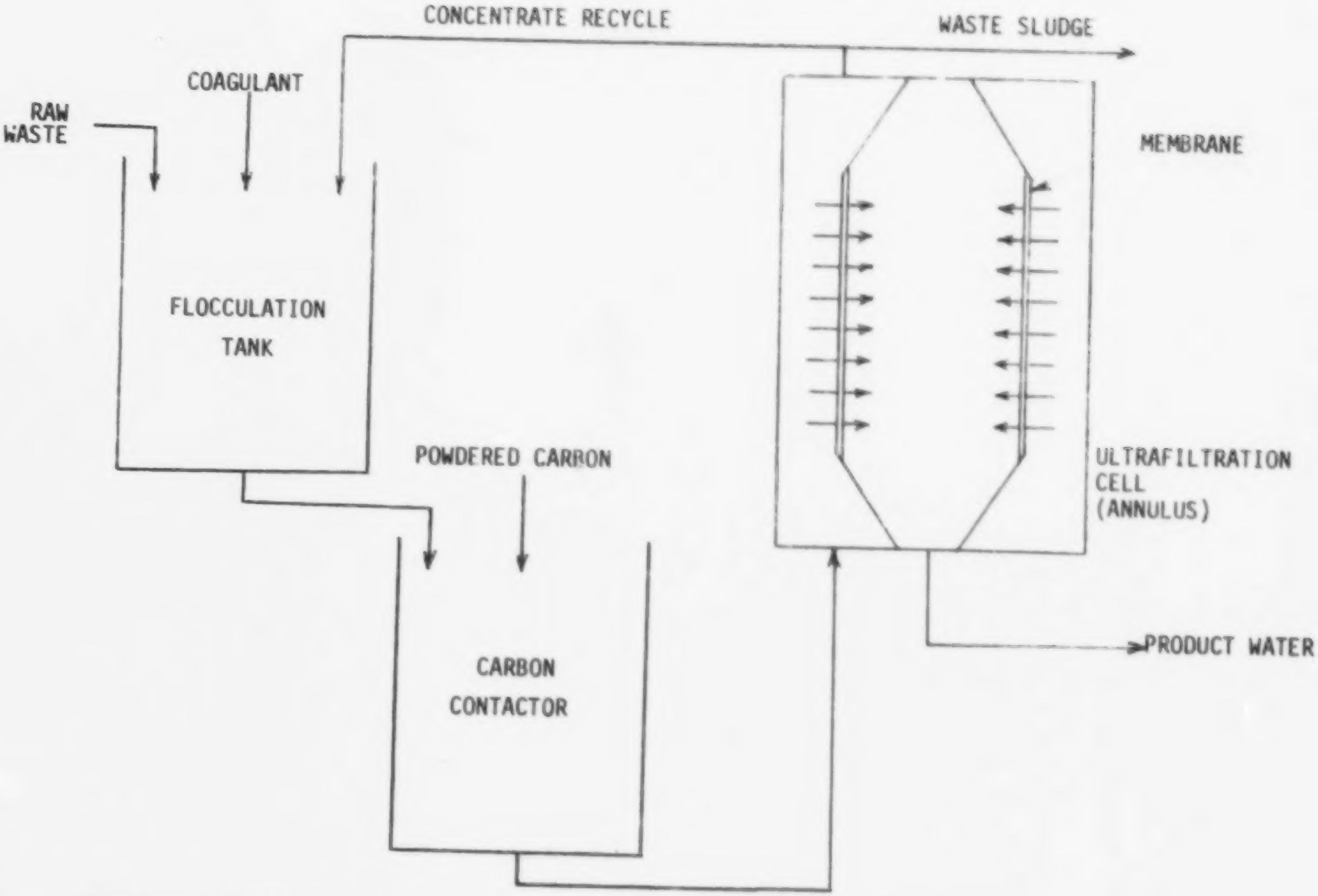
The membrane in this plant is more unique. It is cast on a very coarse membrane support in the form of a firehose jacket on a stainless steel porous plate. The membrane itself is formed dynamically of a film of sewage constituents and filter-aid (here the coagulant). The dynamic forming process must also be closely watched since too heavy a membrane film will produce too tight a membrane. This in turn would reject a high fraction of the salts and produce a correspondingly lower flux (and greater flux decline). Given proper hydrodynamics, the membrane flux decline may be reasonable. Such a membrane has the advantage of negligible cost and ease of replacement, since it can be formed and destroyed in place. This unusual small Advanced Waste Treatment plant is very experimental to date, but holds promise of great versatility for a wide variety of pollutants.

Summary

1. An urgent need exists for "small flow" treatment plants for a variety of situations.

2. Treatment systems based on Advanced Waste Treatment processes are far superior to biological package plants for small flow situations because of their stability of operation in the face of biologically toxic materials, and significant load variations. In addition, reuseable water if required can result from Advanced Waste Treatment processing.

3. Three general Advanced Waste Treatment based schemes of utility for "small flows" have been discussed. The three schemes involve a) clarification-carbon, b) ultrafiltration, c) chemical-biological processing.


CONCENTRATE RECYCLE
WASTE SLUDGE
COAGULANT
RAW WASTE
MEMBRANE
FLOCCULATION TANK
POWDERED CARBON
ULTRAFILTRATION CELL (ANNULUS)
CARBON CONTACTOR
PRODUCT WATER

SCHEMATIC FLOW DIAGRAM OF A CLARIFICATION-POWDERED CARBON-ULTRAFILTRATION SYSTEM
(AFTER OAK RIDGE NATIONAL LABORATORY)
FIGURE 13

4. Four specific systems which may have utility as "small flow" treatment plants have been described in some detail. Preliminary treatment performance from three is discussed.

5. Only one of these was specifically designed as a "small flow" plant and probably none are ideal for such a purpose.

6. Extensive development and testing will be conducted in the future to meet the needs for small flow plants.

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ADVANCED WASTE TREATMENT AND

WATER REUSE SYMPOSIUM

HIGH PERCENTAGE OXYGEN AERATION
FOR SECONDARY TREATMENT

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Federal Water Quality Administration
Ohio Basin Region
Cincinnati, Ohio

HIGH PERCENTAGE OXYGEN AERATION FOR SECONDARY TREATMENT

Richard C. Brenner

INTRODUCTION

The concept of using oxygen enriched air or pure oxygen as the aeration gas in the activated sludge process has intrigued investigators for over twenty years. In 1948, Pirnie (1) suggested a method of predissolving oxygen in wastewater prior to activated sludge treatment. The following year, Okun expanded on Pirnie's model and developed his "bio-precipitation system" which utilized an integral upflow bio-reactor and clarifier (2). Later, Budd and Lambeth (3) evaluated Okun's "bio-precipitation system" in a large pilot plant and reported an oxygen absorption efficiency of 20-25% with power requirements equal to the conventional diffused air technique.

The inherent characteristic of oxygen gas which has created so much interest over the years is its high partial pressure. At atmospheric pressure, the partial pressure of pure oxygen gas is 760 mm of mercury while the partial pressure of oxygen in air is only 160 mm of mercury. Thus, the driving force for dissolving oxygen in water is 4.75 times higher with pure oxygen.

In a "state-of-the-art" review, McKinney and Pfeffer (4) concluded that if the potential of high percentage oxygen aeration could be harnessed in an economically realistic fashion, the following benefits to the activated sludge process could conceivably be achieved:

1. Reduction of power required per unit of oxygen transferred.
2. Reduction in, or elimination of, periods of zero dissolved oxygen concentration.
3. Increased rate of stabilization of organic material.
4. Ability to operate high rate systems with substantial increases in organic loading where oxygen is not limiting.
5. Increased capacity of organically overloaded plants without need for additional aerator capacity.
6. Reduction in plant size and capital investment.

Until recently, however, serious consideration of oxygen aeration has not progressed past the laboratory and pilot plant stages. The basic problem has been the lack of an effective oxygen gas contacting system with a built-in high overall utilization factor. Since oxygen is an expensive commodity and since conventional aeration techniques would vent the major fraction of the input oxygen to the atmosphere, the widespread impression has been that oxygen aeration must, of necessity, be significantly more costly than air aeration. This rationale led Okun and Lynn (5) to conclude in 1956 that the substitution of oxygen for air or the use of oxygen enriched air in standard activated sludge aeration tanks would not be economically feasible because of the poor oxygen transfer efficiency inherent in diffuser and tank design.

Okun and Lynn's position as stated above has proven to date to be partially correct. Certainly, the requirements for an economical oxygen contacting system are such that conventional air aeration technology is entirely inadequate. However, what Okun and Lynn did not forsee was the covering of standard activated sludge tanks to form a gas tight structure. An efficient oxygen aeration system (which will be discussed at length in the following sections) has been developed which utilizes covered longitudinal aeration tanks. The development of an efficient open tank oxygen aeration system also appears to be within the realm of feasibility.

STIMULATION OF RENEWED INTEREST IN OXYGEN AERATION

In early 1968, the Linde Division of Union Carbide approached the FWQA concerning a new oxygen gas contacting concept developed in their laboratories. According to Union Carbide, this concept satisfied all the basic requirements of a cost effective oxygen dissolution system:

1. High percentage oxygen absorption and utilization (i.e. >90%).
2. High overall energy transfer efficiency as compared with air aeration systems.
3. Efficient liquid mixing sufficient to maintain uniform mixed liquor composition.
4. Simple design with low maintenance costs.
5. Low capital investment.

Continuing discussions culminated in the awarding of a $528,000 FWQA Research and Development Contract to Union Carbide in October, 1968. The contract was carried out at the Batavia, New York Treatment Plant. Under the terms of the contract, the performance and economics of the

Linde oxygen aeration (oxygenation) system were evaluated and compared with those of a parallel air aeration system from May to November of 1969.

CONFIGURATION AND OPERATION OF OXYGENATION SYSTEM

A schematic respresentation of the Union Carbide oxygenation system is shown in Figure 1. A gas tight cover on top of the aeration tank is utilized to prevent oxygen leakage to the atmosphere. Baffles are installed to form a series of staged comartments down the length of the aerator. Each compartment is equipped with a submerged turbine-rotating sparger combination. A recirculating gas compressor is located on top of or adjacent to each compartment.

Liquid waste and oxygen gas flow are cocurrent with both being introduced to the head of the aeration tank. Recycle sludge also enters at this point. Oxygen gas fed to the first compartment is compressed and forced down the hollow shaft of the submerged turbine and out through the rotating sparger. The fine bubbles thus created are dissolved in the mixed liquor. The propeller blades furnish bulk fluid mixing and create a downward pumping action which provides longer residence time for the bubbles.

Oxygen which is not utilized by microorganisms vents back into the head space of the first compartment. Here it is either compressed and recirculated again or, due to the slight positive pressure of 1-3" water column maintained in the gas phase, moves through an appropriately sized opening in the baffle wall to be used in the second compartment by the same technique. In like fashion, vent gas from each compartment is used in the succeeding compartment until the end of the aeration tank is reached. A small exhaust stream (10-20% of the input gas stream) vents to the atmosphere from the last compartment. It should be pointed out that the gas stream is picking up reaction products and inert gases as it proceeds down the compartments. The exhaust gas is, therefore, about 50% oxygen, and 50% carbon dioxide, nitrogen, argon, etc. The small exhaust rate and the reduced composition of oxygen in the exhaust gas combine to yield an overall oxygen utilization of 90-95%.

SCOPE OF THE BATAVIA PROJECT

The nominal capacity of the Batavia plant is 2.5 mgd. As indicated in Figure 2, the plant consists of two parallel and identical 1.25 mgd aeration tanks each with its own final clarifier and separate return sludge line. Each aeration tank has two passes. There are no primary settlers at Batavia. Raw wastewater is screened, degritted, and comminuted before entering the aerators. Waste activated sludge is handled in sequence by gravity thickening, two stage anaerobic digestion, vacuum filtration, and trucking to disposal. Treated effluent is chlorinated before discharge to Tonawanda Creek.

The oxygenation system was installed in one of the two aeration tanks. A temporary steel deck cover was bolted and sealed to the top of the oxygen aerator for convenience and economy. Baffle installations were also designed for short term use. Six compartments or stages were utilized, three in each pass. Oxygen was stored on site in the liquid form and then vaporized before introduction to the oxygen aerator. Oxygen feed rate to the first compartment was automatically controlled by maintaining a constant vapor pressure in the gas space of 1-3" of water. Dissolved oxygen level was also automatically controlled, but in the first compartment only where oxygen demand was highest. This was accomplished with a dissolved oxygen probe and a feedback signal which controlled the rate of recirculation of the first stage compressor.

The existing diffused air system was left intact in the second aerator for the comparative studies. Aeration capacity of this train was increased with 50% additional laterals and diffusers. Compressed air was supplied to the air train by the plant's rotary blowers.

The experimental program was segmented into three phases; a conservative phase, a medium stressed phase, and a highly stressed phase. To take advantage of dry weather flow, the highly stresed phase (Phase II) was conducted in the summer immediately after the conservative phase (Phase I). The medium stressed phase (Phase III) was conducted last. The modes of operation of the three phases are given in Figures 3, 4, and 5.

In Phase I, the average plant flow of 3.88 mgd was divided evenly as nearly as possible between the two trains. All six compartments of the oxygen train and both passes of the air train were utilized. The intended aeration detention time in each train based on raw flow was 6 hours, but the high spring flows reduced this to about 4 hours. Seven weeks of comparative data were collected in this phase.

All of the plant flow was directed to the oxygen train in Phase II. To further stress the system, three of the six stages were taken out of service. Both clarifiers and air lift pumps were utilized to separate and return the sludge. The intended and actual aeration time averaged 1.5 hours. Seven weeks of data were also collected in this phase.

The plant flow was again split between the two trains in Phase III. Three of the six oxygen compartments and one pass of the air train were taken out of service to achieve a medium loaded condition. The intended aeration detention time of 3 hours was realized with the air train. The detention time of the oxygen train averaged somewhat lower at 2.8 hours. Nine weeks of comparative data were collected in this phase.

SUMMARY AND DISCUSSION OF BATAVIA PERFORMANCE DATA

Comparative performance data generated on the Batavia project are summarized in Figures 6 through 9. Each of the data figures will be discussed in order below:

Figure 6: Excellent operation and surveillance were given to both systems throughout. The degree of operator attention was 2-3 times that normally expended at treatment plants. Well-trained Union Carbide technicians operated the experimental facilities during the week; Batavia plant personnel were utilized on the weekends.

It was decided to operate the air train in Phase I at a mixed liquor suspended solids (MLSS) concentration typical of other conventional air systems of approximately 2500 mg/l. MLSS in the oxygen train in Phase I were kept at 40-50% of the maximum concentration which it was anticipated could be carried in later phases, or about 3000 mg/l. In order to treat effectively under the high rate loading conditions of Phase II, oxygen train MLSS were raised to the maximum sustainable level of 7000 mg/l. This upper limit was not due to oxygen transfer limitations but to restricted clarification capacity. In Phase III, oxygen train MLSS were kept high at about 6200 mg/l while air train MLSS were raised to the maximum point (3700 mg/l) at which some measurable dissolved oxygen (DO) could still be maintained in the mixed liquor. Above 3700 mg/l, DO fell to zero and septic odors commenced.

Sludge settled very well in both systems. Sludge volume index (SVI) varied from 35-65 for the oxygen system and 65-75 for the air system. In both systems, SVI decreased with increasing MLSS. In all probability, the extremely good settling characteristics were due in part to the absence of primary sedimentation.

Recycle sludge concentration was also higher than in most plants. For the air system, the concentration ranged from 1.5-1.6%. The range for the oxygen system was 1.8-3.0%. Recycle sludge concentrations in the oxygen system as high as 4% were noted in Phase II during the day. The installation of a larger flow meter in the oxygen train sludge recycle line to handle the return from both clarifiers in Phase II adversely affected oxygen slydge recycle concentration in Phase III. Return flow had to be kept artificially high in Phase III to maintain sufficient head loss to operate the oxygen air lift pump, thereby thinning out the sludge. It is felt that with more conservative overflow rates and variable speed return pumps, oxygenated sludge can be thickened routinely to 3-4% in the final clarifier. At these concentrations, it is doubtful if the waste oxygenated sludge would have to undergo separate thickening before digestion or direct vacuum filtration.

Mixed liquor DO averaged 8-9 mg/l in the oxygen train in all phases. Variations in DO profile down the stages were insignificant. The low oxygen effluent DO in Phase II indicates that some treatment was still going on in the final clarifier due to the high organic loading. Air train mixed liquor DO was low in both Phases I and III. No measurable DO was present at the head of the air aerator in either phase, despite abnormally high system air input rates of 3-4 ft^3/gal of sewage treated. The poor oxygen transfer efficiency of disc diffusers is readily apparent.

Figure 7: As anticipated, BOD and suspended solids (SS) removals were good for both systems in the conservative Phase I. Effluent COD values of 73 mg/l for the oxygen train and 84 mg/l for the air train are considered acceptable for aerators treating unsettled sewage. Volumetric organic loadings were in the upper portion of the range for which most conventional plants are designed. Overall performance in Phase I was slightly better for the oxygen train.

One of the most encouraging aspects of the entire project was the acceptable level of performance of the oxygen system under the high rate conditions of Phase II. Effluent BOD, COD, and SS values were not as low as in Phase I but still within the range of good secondary treatment. The volumetric organic loading was 3-1/2 times higher than in Phase I. The ability of the oxygen system to operate satisfactorily under much higher organic loading rates than normally associated with air systems was clearly established in this phase.

Wastewater feed strength was at its highest in Phase III. Some problems were encountered with the flow splitting device to the two aerators with the result that about 10% more flow was directed to the oxygen train. Performance of the oxygen train was excellent. Percent removals of BOD, COD, and SS were the highest of any of the three phases although absolute effluent concentrations were slightly higher than in Phase I. The air system began to show signs of performance deterioriation in Phase III. There was zero DO in the air train effluent. Effluent BOD and SS concentrations were about double and effluent COD 30% higher than those of the oxygen system. Volumetric organic loadings were more than twice as high as in Phase I.

Somewhat unexpected was the magnitude of the reduced quantities of waste activated sludge generated throughout the study in the oxygen system. Disregarding the data point for the oxygenation system in Phase III (which is extremely out of balance with the other two phases), 30-40% less sludge was wasted from the oxygen system than from the air system. Considering that sludge handling and disposal account for 1/2 to 2/3 of total annual treatment costs, this factor assumes significant economic importance. From the above data, it is evident that oxygenation is a highly oxidative process.

Figure 8: The two principal conclusions which can be drawn from these data are that phosphorus removal was in the range normally associated with biological uptake for both systems and that neither system nitrified. For the two comparative phases, phosphorus removal averaged 5% higher for the air system. This is to be expected as the oxygen system "burned up" more sludge and thereby, in all likelihood, solubilized greater quantities of stored cellular phosphorus. Only the very beginnings of incipient nitrification were noticed in either unit. This was the one disappointing feature of the Batavia oxygen train performance, notwithstanding that prevailing temperatures and organic loading rates were not exactly conducive to nitrification.

Figure 9: High percentage utilization of feed oxygen was achieved at greater than 90% in each phase. The average composition of oxygen in the exhaust gas for the project was 51%.

Power consumption for oxygen dissolution is broken into two categories, liquid mixing and gas recirculation. Power consumption for liquid mixing is roughly proportional to the gallons of mixed liquor under oxygenation while gas recirculation power is highly dependent on waste strength and the total pounds of BOD or COD entering the system. It should be pointed out that the oxygen compressors used on this project were substantially oversized for experimental flexibility and that the power requirements shown here for gas recirculation represent only that fraction of gas which was actually put through the turbine shaft. Power consumed in bypassing unneeded gas is not included. This is a legitimate technique as in a non-experimental situation, compressors are sized more closely to design conditions and variable speed compressors are used in the high oxygen demand stages.

Abnormally high aeration rates were required in the air train in an attempt to cope with the high organic strength of the unsettled wastes. This resulted in an air blower power requirement of 5-6 times that for oxygen dissolution. For a true comparison of total power requirements, however, the power consumed in on-site oxygen gas production should be added to that used in oxygen dissolution. Since liquid oxygen was used in this project, no such totals are possible here. In general, though, power requirements for oxygen generation will range from 1.3 to 1.7 times greater than the power required for oxygen dissolution. Even so, a considerable reduction in overall power costs with oxygen is indicated.

ECONOMIC PROJECTIONS

The following assumptions are used in conjunction with the Batavia data in arriving at comparative cost curves for the two systems:

1. Raw sewage pumping and metering, preliminary treatment (including screening, comminution, and grit removal), primary sedimentation, service and administration buildings, and outside piping and appurtenances are costed in equally for both systems.

2. Raw wastewater BOD is assumed to be 200 mg/l with 35% removed in the primary settlers.

3. Equal amounts of primary sludge are produced in each system at the rate of 860 lb/mil gal treated. Waste activated sludge quantities for the two systems are in accordance with Figure 10 which was derived from the Batavia data. Phase III oxygen system waste sludge data were not used in constructing Figure 10.

4. MLSS concentration for the oxygen and diffused air systems are assumed to be 6800 mg/l and 2500 mg/l, respectively, with 65% of the sludge being volatile in each case.

5. Based on assumption 4 and using Figure 11 which was compiled from the Batavia work, 1 hour of oxygen aeration will provide equivalent treatment to 3 hours of air aeration.

6. For each size plant costed, four aeration detention points (based on raw flow) are plotted and compared. These points are 1.0, 1.33, 1.66, and 2.0 hours for oxygen and 3.0, 4.0, 5.0, and 6.0 hours for air. Percent BOD removal associated with each set of comparative points is given in Figure 12. Volumetric organic loadings and food to biomass ratios for the various aeration detention times are shown in Figure 13.

7. Due to the variance in oxygen transfer capability of different types of air diffusers, three air aeration rates are costed for the air system including 0.8, 1.6, and 2.4 ft^3/gal wastewater treated.

8. Oxygen supply varies from 1.0 to 1.22 lb/lb BOD removed as oxygenation detention time increases from 1.0 to 2.0 hours. Average oxygen utilization is 91%.

9. Mixed liquor DO is assumed at 8 mg/l for the oxygen system and 1 mg/l for the air system.

10. Sludge recycle oncentrations are assumed to be 40,000 mg/l and 10,000 mg/l, respectively, for oxygen and air systems.

On these bases, average sludge recycle rates in the same order are 20% and 33% of influent flow. Spare recycle pump capacity of 100% average recycle flow is costed in for each system.

11. Oxygen is supplied to the oxygenation system by on-site generation of oxygen gas. For plant sizes up to approximately 30 mgd, the generation plant would be of the air separation type. Above 30 mgd, a cryogenic plant is used. Union Carbide has made a managerial decision that municipalities, if they so choose, may purchase the plant outright rather than purchasing liquid or gaseous oxygen across the fence. The savings to the municipality in $/ton of oxygen by buying the plant instead of the commodity itself are evident in Figure 14. The bottom curve is used in the cost projections given herein.

12. Total oxygenation system and air aeration system power requirements for the four plant sizes (1, 6, 30, and 100 mgd) used later in the comparative cost curves are indicated in Figures 15 and 16, respectively. Power requirements for the oxygen system are a function of oxygenation detention time; those for the air system are a function of air aeration rate. Reserve air system blower capacity of 100-25% is provided as plant size increases from 1 to 100 mgd. Backup capacity for the oxygen generators includes a spare air compression unit for the 1, 6, and 30 mgd plant sizes and a 24-hour on-site supply of liquid oxygen for the 100 mgd plant.

13. Sludge disposal for the 1 mgd plant is assumed to be by anaerobic digestion followed by sludge drying beds, for the 6 mgd plant by anaerobic digestion, vacuum filtration, and cake disposal by land fill, and for the 30 and 100 mgd plants by anaerobic digestion, vacuum filtration, and incineration. Both primary and waste activated sludges were costed into the sludge disposal estimates.

14. A gravity thickener is costed in for the air system to thicken waste activated sludge from 1 to 4% prior to anaerobic digestion. Since it is assumed that waste activated sludge is drawn from the oxygen system at 4%, a thickener is not needed for the oxygen system.

15. Secondary clarifiers are sized at 800 gal/day/ft^2 for both systems at a liquid depth of 10 ft for the air system and 11 ft for the oxygen system.

16. Cost curves and formulae for construction and operation of conventional plant units and for sludge thickening and disposal were supplied to Union Carbide by FWQA. Cost data for oxygen generation, dissolution, and control equipment and for oxygenation tank covers and baffles were furnished by Union Carbide. Amortization was calculated at 5-1/2% over 25 years. Construction costs were determined using January, 1970 indices.

The above assumptions have been compiled into four comaparative cost estimate curves for 1, 6, 30, and 100 mgd plant sizes. These cost estimates are given in Figures 17 through 20 as total treatment costs in ¢/thousand gallons treated. The costs reflect debt retirement and operation and maintenance of all plant facilities including sludge disposal equipment, i.e. the total cost associated with running a combined primary-secondary treatment plant with the exception of effluent chlorination.

As can be seen in Figure 17, there is no cost advantage with oxygen at 1 mgd. At the 6 mgd scale, there is an estimated cost savings with oxygen ranging from 11-15% when compared with an air aeration rate of 0.8 ft^3/gal and 17-21% when compared with an air aeration rate of 1.6 ft^3/gal. For the 30 mgd plant, estimated cost reductions with oxygen range from 16-20% and 21-27% for the comparisons with 0.8 and 1.6 ft^3/gal, respectively. At 100 mgd, the estimated cost reductions with oxygen are 17-25% and 22-29% compared to the same air aeration rates, respectively.

It is emphasized that the cost data shown in these figures are estimates only and that individual plant costs may vary widely with locale. Further, it is realized that air aeration equipment vendors, particularly those that manufacture surface and turbine aerators, may disagree with the air system design used in the cost comparisons. Nevertheless, it is believed the air system design so presented reflects conventional plug flow designs as they are approved by state regulatory agencies and used in the majority of the municipalities in this country. Likewise, the oxygen system design presented herein reflects the manner in which it is contemplated such a system would be designed for a municipal installation.

APPLICATIONS OF THE OXYGEN AERATION PROCESS

It is felt the results of the Batavia project justify consideration of the oxygen aeration process for all new secondary plants and primary plants expanding to secondary treatment. In this type of application, the City of Detroit, Michigan has solicited bids with State of Michigan and FWQA Construction Grants approval for a 300 mgd oxygen aeration module for its new METRO plant. Also, the State of New York has approved plans for a 15 mgd oxygen aeration system at New Rochelle, New York.

The oxygenation process also appears to be equally applicable to upgrading existing overloaded activated sludge plants. It is believed that the aeration capacity of many overloaded systems can be increased 2-3 times without acquiring additional land for more aeration tanks. To test this hypothesis, the FWQA has recently awarded a $1.5 million Demonstration Grant to New York City for conversion of one existing 20 mgd bay of its Newtown Creek Plant to oxygen. The plant is currently accomplishing about 40% BOD removal with 1-1/2 hours of air aeration. It is hoped the removal efficiency of the test bay will be improved to 90% with oxygen. The test bay is scheduled to be on stream by late fall, 1971, and the grant evaluation will then continue for one year.

Other cities where Union Carbide is conducting non-federally sponsored pilot plant studies include Cincinnati, New Orleans, Louisville, Euclid, Ohio, Middlesex, New Jersey, and Speedway, Indiana.

AREAS NEEDING FURTHER INVESTIGATION

Questions remain to be answered concerning the following aspects of aeration with oxygen:

1. Nitrification: As mentioned previously, the Batavia experiments did not demonstrate nitrification. Definite criteria need to be established at which nitrification with oxygen can be guaranteed. Experiments are currently underway at the FWQA Blue Plains pilot plant in which a two-step oxygenation flow regime with intermediate clarification is being utilized to study this problem.

2. Sludge handling and disposal: The high solids concentrations and reduced quantities of waste activated sludge generated with oxygenation suggest several alternative approaches to anaerobic digestion for sludge disposal. Because of the highly oxidized condition of the sludge, several days of aerobic digestion may be sufficient to stabilize the sludge for land spreading. It may also be possible to dewater waste sludge directly from the clarifier underflow without any type of prior digestion. A follow-on contract for $208,714 was awarded to Union Carbide for further studies at Batavia during the summer and fall of 1970, a part of which included direct vacuum filtration of waste oxygenated sludge, aerobic stabilization of waste oxygenated sludge, and vacuum filtration of aerobically stabilized sludge. The final report from this follow-on contract will be available around May, 1971.

3. Carbon dioxide buildup in mixed liquor: Because of the closed aeration system, carbon dioxide (CO_2) tends to build up in solution in an oxygen aerator. At Batavia, this caused a drop

in mixed liquor pH of 0.5-0.7 units in all phases. Thus, it can be speculated that for stronger wastes more CO_2 will be produced and the drop in pH could be large enough to endanger the sludge culture.

Air Reduction Company is currently conducting a pilot study at the Greensboro, North Carolina South Buffalo Plant. In addition to domestic wastes, this plant receives all of the City's textile mill wastes. Organic strength of the combined municipal wastewater is very high. The Air Reduction oxygen aeration system uses a large diameter PVC pipe reactor to replace the standard open or closed aeration tank. Oxygen injection to the pipeline can be single or multi point as needed. The advantage of this system is that it requires no moving parts other than the prime mover for oxygen dissolution. If the pilot studies indicate the high strength wastewater can be effectively treated with oxygen, it is contemplated that the City of Greensboro will apply to FWQA for a Demonstration Grant, during which CO_2 buildup would be further investigated.

4. Effect of primary sedimentation: The effect of primary sedimentation on the oxygen process, particularly on sludge settling characteristics is not known. At this time, a recommendation for or against primary sedimentation in conjunction with oxygen aeration can not be made. The Newtown Creek Plant does not have primary settlers, so the answer to this question will have to come from privately funded studies or future federal grant projects.

CONCLUSIONS

1. Oxygen aeration is a highly oxidative process which operates effectively as a high rate system.

2. Significant reductions in waste activated sludge production are indicated with oxygen aeration.

3. Oxygen sludge settles and thickens well. It may be possible to dewater waste sludge from this process directly from the clarifier underflow.

4. The oxygen aeration process is applicable to both new and existing overloaded activated sludge plants.

5. Cost reductions in total treatment costs of 15-30% appear feasible for new plants with oxygen. Percent savings will be greater for conversion of existing overloaded plants to oxygen.

6. Several aspects of the oxygen aeration process require further investigation to maximize its full potential for waste treatment.

APPENDIX

All of the figures utilized in this discussion are reproduced directly from the final report of the Batavia project (6). Copies of this report can be obtained by writing to either of the following:

Planning and Resources Office
Office of Research and Development
Federal Water Quality Administration
U. S. Department of the Interior
Washington, D. C. 20242

Advanced Waste Treatment Research Laboratory
Federal Water Quality Administration
U. S. Department of the Interior
4676 Columbia Parkway
Cincinnati, Ohio 45226

Attention: Mr. R. C. Brenner

REFERENCES

1. Pirnie, M., Paper presented at 21st Annual Meeting of Federation of Sewage Works Associations, Detroit, Michigan (October 1948).

2. Okun, D. A., "System of Bio-Precipitation of Organic Matter from Sewage," Sewage Works Journal, 21 (1949).

3. Budd, W. E. and Lambeth, G. F., "High Purity Oxygen in Biological Sewage Treatment," Sewage and Industrial Wastes, 29, 253 (1957).

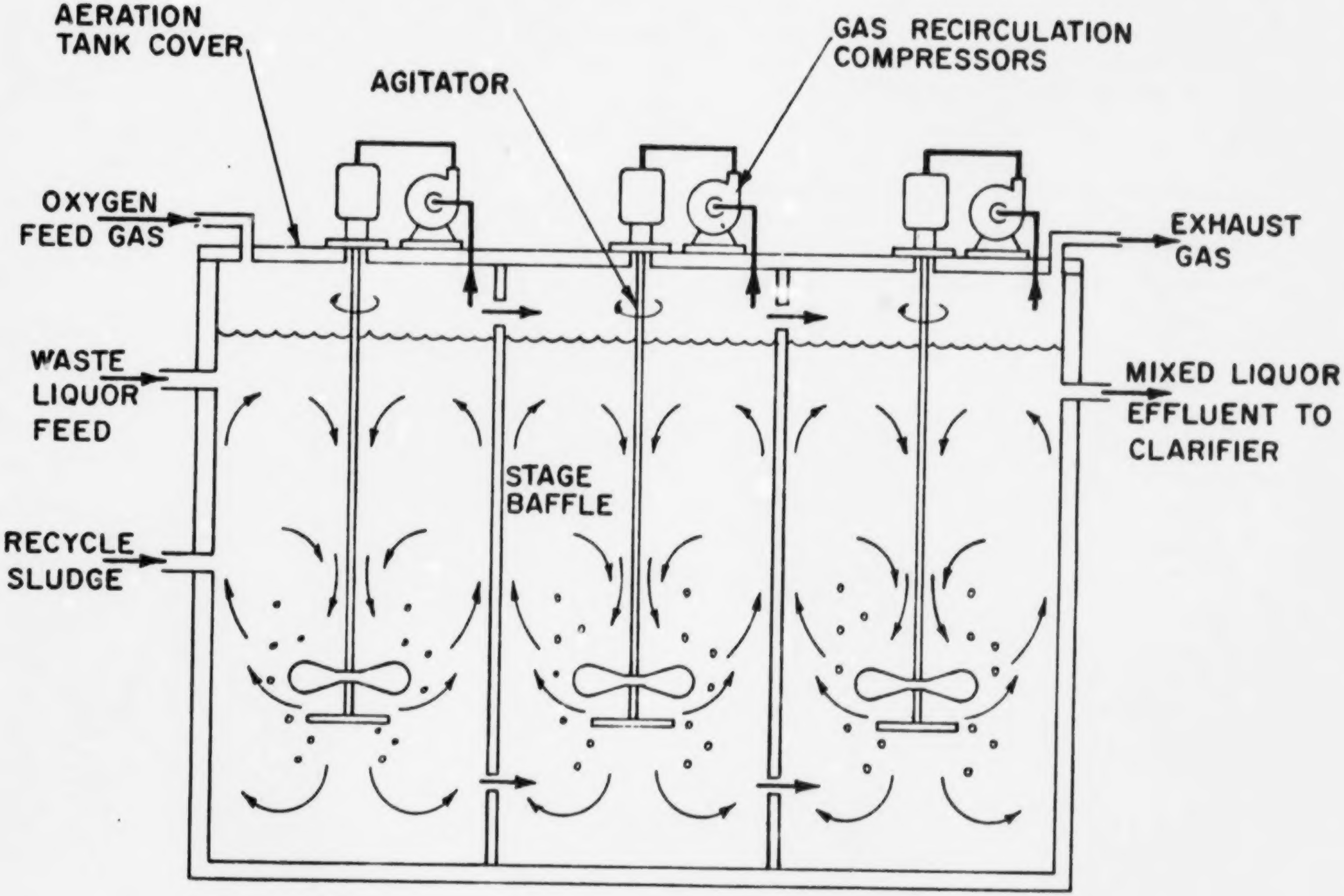
4. McKinney, R. E. and J. T. Pfeffer, "Oxygen Enriched Air for Biological Waste Treatment," Water and Sewage Works (October 1965).

5. Okun, D. A. and Lynn, W. R., "Preliminary Investigation into the Effect of Oxygen Aeration on Biological Sewage Treatment," Biological Treatment of Sewage and Industrial Wastes: Volume I, Aerobic Oxidation, Reinhold Publishing Corp., New York (1956).

6. Albertsson, J. G., McWhirter, J. R., Robinson, E. K., and Vahldieck, N. P., "Investigation of the Use of High Purity Oxygen Aeration in the Conventional Activated Sludge Process," FWQA Research Contract No. 14-12-465, Program No. 17050 DNW (May 1970).

FIGURE I

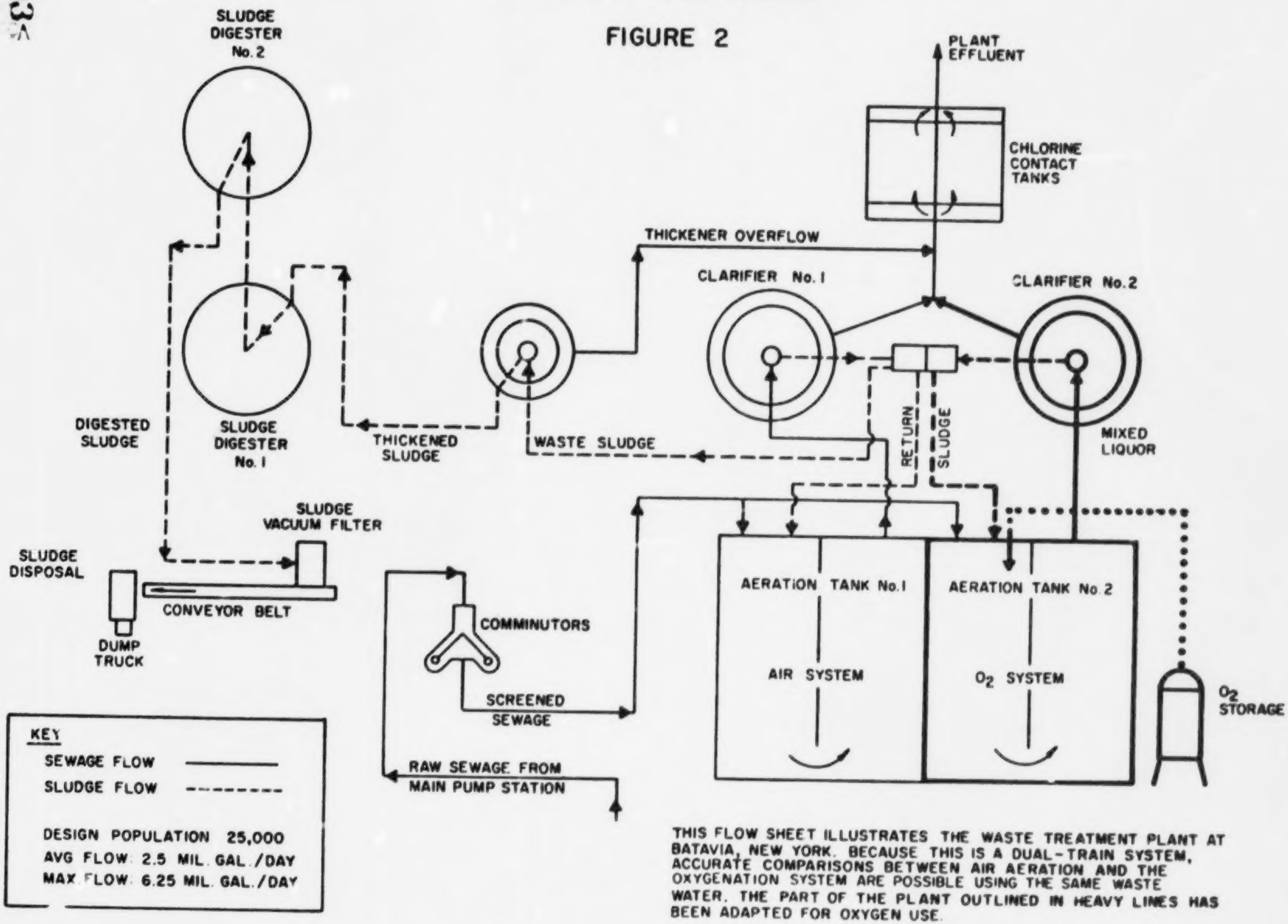
SCHEMATIC DIAGRAM OF MULTI-STAGE
OXYGENATION SYSTEM


AERATION TANK COVER
GAS RECIRCULATION COMPRESSORS
AGITATOR
OXYGEN FEED GAS
EXHAUST GAS
WASTE LIQUOR FEED
MIXED LIQUOR EFFLUENT TO CLARIFIER
STAGE BAFFLE
RECYCLE SLUDGE

14-6

WATER POLLUTION CONTROL PLANT, CITY OF BATAVIA, N.Y.
SCHEMATIC FLOW DIAGRAM

FIGURE 2


SLUDGE DIGESTER No. 2
SLUDGE DIGESTER No. 1
DIGESTED SLUDGE
THICKENED SLUDGE
WASTE SLUDGE
THICKENER OVERFLOW
PLANT EFFLUENT
CHLORINE CONTACT TANKS
CLARIFIER No. 1
CLARIFIER No. 2
RETURN SLUDGE
MIXED LIQUOR
SLUDGE VACUUM FILTER
SLUDGE DISPOSAL
CONVEYOR BELT
DUMP TRUCK
COMMINUTORS
SCREENED SEWAGE
RAW SEWAGE FROM MAIN PUMP STATION
AERATION TANK No. 1
AIR SYSTEM
AERATION TANK No. 2
O2 SYSTEM
O2 STORAGE
KEY
SEWAGE FLOW
SLUDGE FLOW
DESIGN POPULATION 25,000
AVG FLOW 2.5 MIL. GAL./DAY
MAX FLOW 6.25 MIL. GAL./DAY

THIS FLOW SHEET ILLUSTRATES THE WASTE TREATMENT PLANT AT BATAVIA, NEW YORK. BECAUSE THIS IS A DUAL-TRAIN SYSTEM, ACCURATE COMPARISONS BETWEEN AIR AERATION AND THE OXYGENATION SYSTEM ARE POSSIBLE USING THE SAME WASTE WATER. THE PART OF THE PLANT OUTLINED IN HEAVY LINES HAS BEEN ADAPTED FOR OXYGEN USE.

14-c

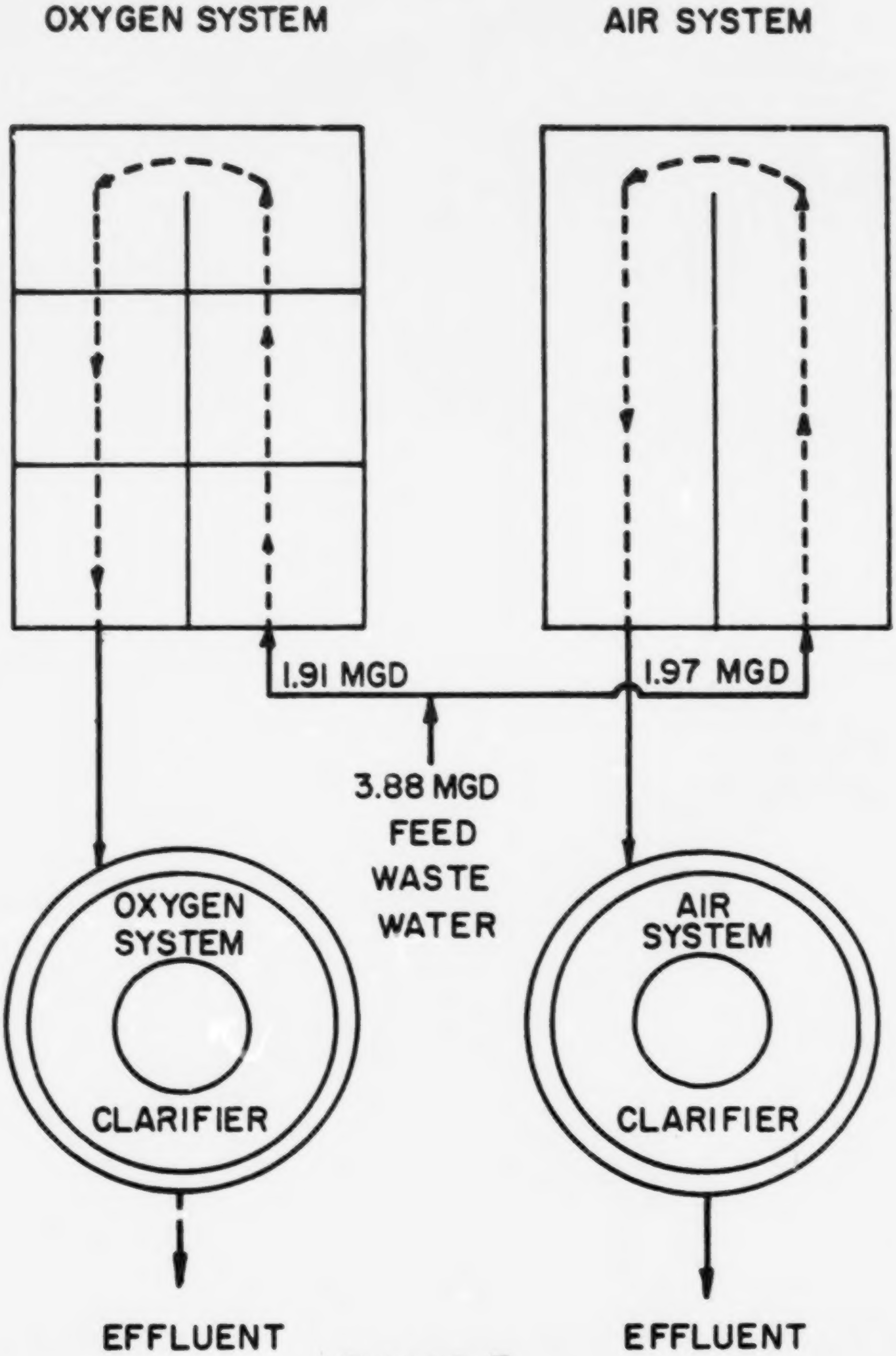

OXYGEN SYSTEM
AIR SYSTEM
1.91 MGD
1.97 MGD
3.88 MGD FEED WASTE WATER
OXYGEN SYSTEM CLARIFIER
AIR SYSTEM CLARIFIER
EFFLUENT
EFFLUENT

FIGURE 3

BATAVIA TREATMENT SYSTEM

PHASE * I OPERATION

-14-d

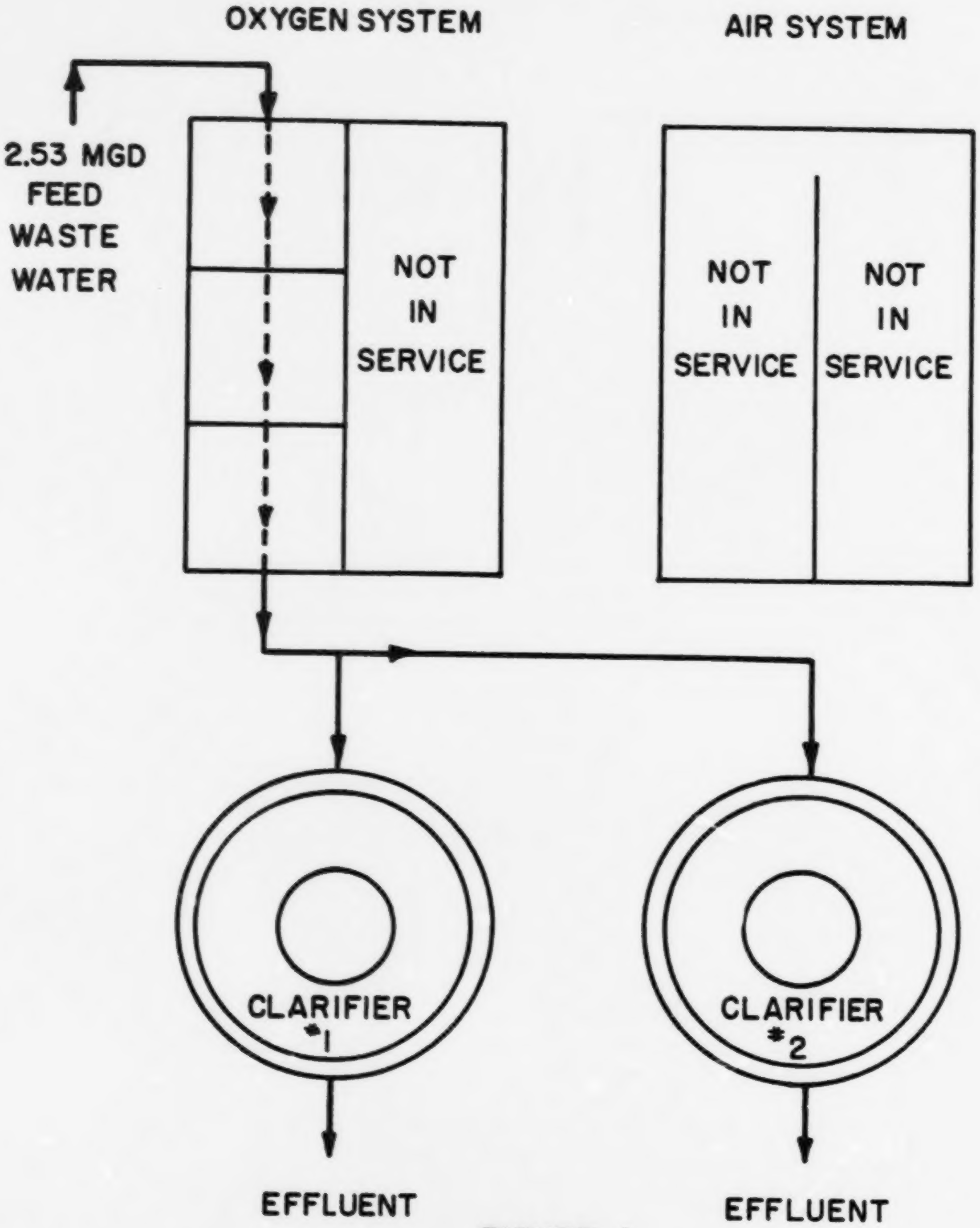

OXYGEN SYSTEM
AIR SYSTEM
2.53 MGD FEED WASTE WATER
NOT IN SERVICE
NOT IN SERVICE
NOT IN SERVICE
CLARIFIER *1
CLARIFIER *2
EFFLUENT
EFFLUENT

FIGURE 4

BATAVIA TREATMENT SYSTEM
PHASE *2 OPERATION

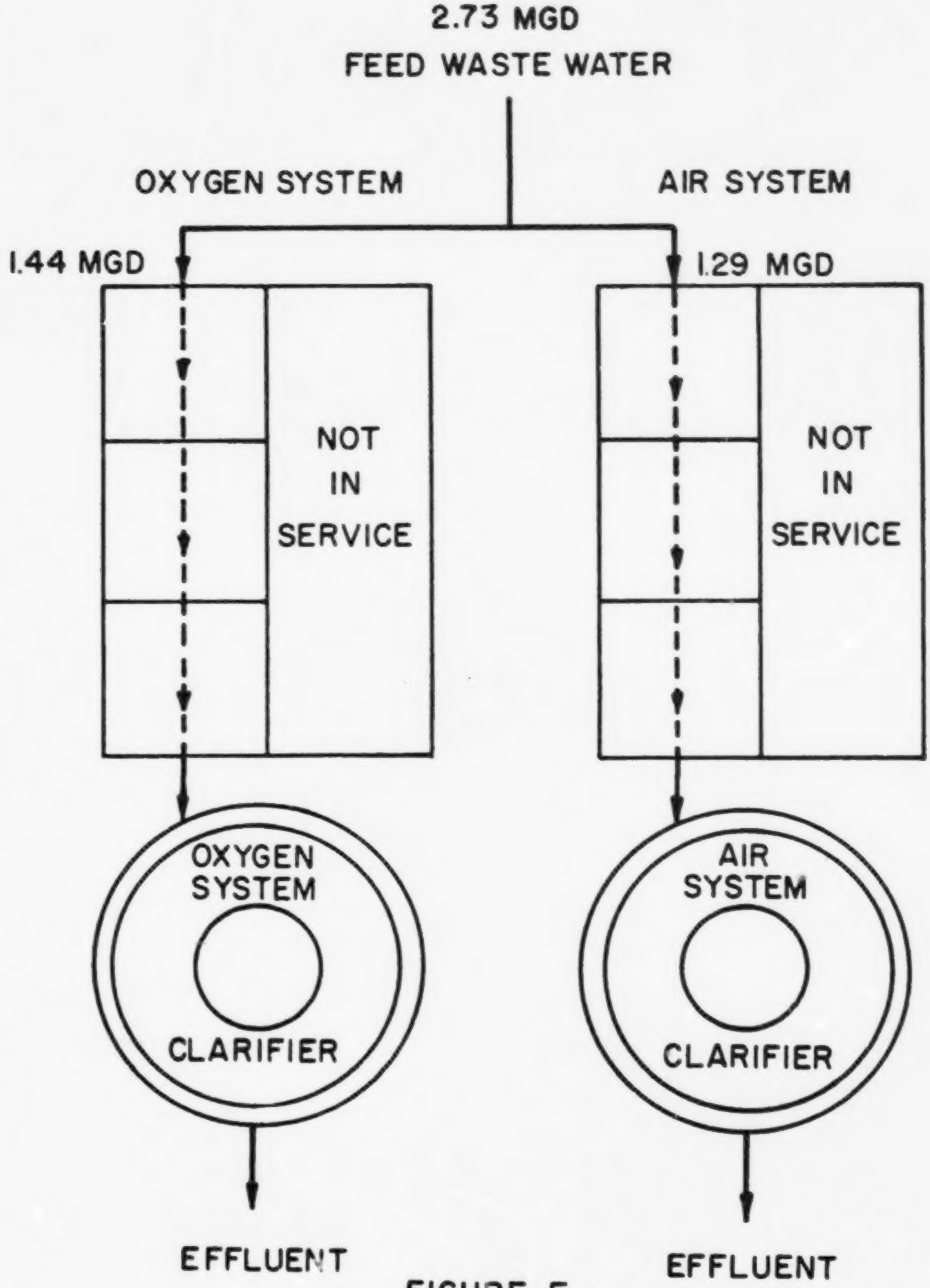

2.73 MGD
FEED WASTE WATER
OXYGEN SYSTEM
AIR SYSTEM
1.44 MGD
1.29 MGD
NOT IN SERVICE
NOT IN SERVICE
OXYGEN SYSTEM
CLARIFIER
AIR SYSTEM
CLARIFIER
EFFLUENT
EFFLUENT

FIGURE 5

BATAVIA TREATMENT SYSTEM

PHASE # 3 OPERATION

14-f

FIGURE 6

COMPARATIVE SUMMARY OF AIR AND OXYGENATION TREATMENT SYSTEM PERFORMANCE

	Phase I Operation		Phase II Operation	Phase III Operation	
	Air System	Oxygenation System	Oxygenation System	Air System	Oxygenation System
Wastewater Feed Rate (MGD)	1.97	1.91	2.53	1.29	1.44
Sludge Recycle Rate (MGD)	0.25	0.44	0.83	0.25	0.60
Recycle Sludge/Wastewater Feed Rate Ratio	0.13	0.24	0.34	0.21	0.45
Wastewater Temperature (°F)	59	59	66	70	70
Mixed-liquor average D.O. Conc.(mg/l)	1.5	8.7	9.0	0.8	8.0
Clarifier Effluent D.O. Conc. (mg/l)	0.4	4.0	0.7	0	4.3
*Aeration Detention Time (hrs.)	3.5	3.4	1.2	2.6	2.0
**Nominal Aeration Detention Time (hrs.)	4.0	4.1	1.5	3.0	2.8
MLSS Concentration (mg/l)	2440	3060	6980	3640	6190
MLVSS Concentration (mg/l)	1740	2210	4450	2580	4310
Recycle Sludge TSS (mg/l)	14,960	18,620	29,560	16,600	18,790
Recycle Sludge VSS (mg/l)	9,710	11,760	18,180	11,840	12,720
Recycle Sludge VSS/TSS Ratio	0.65	0.63	0.62	0.71	0.69
Mixed-liquor Sludge Volume Index	76	64	36	63	49
Mixed-liquor Initial Settling Velocity, (ft/hr)	7.7	7.2	6.5	7.7	5.1
Final Clarifier Overflow Rate as gal/ft^2/day	1570	1520	1010	1030	1140

* Raw flow plus recycle sludge flow.

** Raw wastewater flow only.

GPO 821-871A-7

FIGURE 7

COMPARATIVE SUMMARY OF AIR AND OXYGENATION TREATMENT SYSTEMS PERFORMANCE

	Phase I Operation		Phase II Operation	Phase III Operation	
	Air System	Oxygenation System	Oxygenation System	Air System	Oxygenation System
Feed Wastewater BOD (mg/l)	159	159	220	262	262
Clarifier Effluent BOD (mg/l)	16	11	23	30	14
% BOD Removal	90	92	90	88	94
Feed Wastewater COD (mg/l)	352	352	325	578	578
Clarifier Effluent COD(mg/l)	84	73	97	116	89
% COD Removal	76	80	71	79	84
Feed Wastewater TSS (mg/l)	221	221	174	430	430
Clarifier Effluent TSS (mg/l)	16	9	19	23	12
% TSS Removal	93	96	89	94	97
Feed Wastewater VSS (mg/l)	152	152	123	332	332
Effluent Turbidity (J.T.U.)	5.7	5.6	9.1	5.7	4.4
Food/Biomass Ratio as lbs.BOD/Day/lb. MLVSS	0.566	0.414	0.790	0.841	0.554
Volumetric Organic Loading as lbs.BOD/day/1000 ft^3 mixed-liquor	60.0	57.9	212.5	128.9	144.8
Lbs. Dry Solids Wasted/Day	3148	1804	2448	3494	592
Lbs. VSS Wasted per Lb. BOD Removed	0.871	0.482	0.412	0.989	0.134

14-[illegible]h

FIGURE 8.

COMPARATIVE SUMMARY OF AIR AND OXYGENATION TREATMENT SYSTEMS PERFORMANCE

	Phase I Operation		Phase II Operation	Phase III Operation	
	Air System	Oxygenation System	Oxygenation System	Air System	Oxygenation System
Feed Wastewater Total Phosphate (mg/l)	11	11	12	19	19
Clarifier Effluent Total Phosphate (mg/l)	6	7	10	13	14
% Total Phosphate Removed	44	39	12	32	27
Feed Wastewater NH_3-N (mg/l)	20	20	19	30	30
Clarifier Effluent NH_3-N (mg/l)	17	15	18	24	24
Feed Wastewater TKN (mg/l)	34	34	33	53	53
Clarifier Effluent TKN (mg/l)	22	19	22	31	30
Feed Wastewater NO_2-N (mg/l)	0.2	0.2	0	0	0
Clarifier Effluent NO_2-N (mg/l)	0.4	0.9	0.2	0.2	0.7
Feed Wastewater NO_3-N (mg/l)	0.3	0.3	0.1	0.1	0.1
Clarifier Effluent NO_3-N (mg/l)	0.6	1.6	1.7	2.2	2.5
Feed Wastewater Total Nitrogen (mg/l)	35	35	34	53	53
Clarifier Effluent Total Nitrogen (mg/l)	23	21	26	33	33
% Total Nitrogen Removed	35	37	23	36	36
NO_3-Nitrogen/Total Nitrogen in Clarifier Effluent	0.03	0.08	0.07	0.07	0.07
Lbs. Total Nitrogen Removed per Lb. Dry Solids Wasted	0.066	0.129	0.066	0.060	0.453

FIGURE 9.

COMPARATIVE SUMMARY OF AIR AND OXYGENATION TREATMENT SYSTEMS PERFORMANCE

	Phase I Operation		Phase II Operation	Phase III Operation	
	Air System	Oxygenation System	Oxygenation System	Air System	Oxygenation System
Ft^3 Air Utilized/gal. Sewage Treated	2.89	-	-	4.32	-
Feed Oxygen Flow Rate (cfm,NTP)	-	18.3	31.0	-	19.0
Gas Exhaust Rate from Oxygen System Final Stage (cfm,NTP)	-	1.74	4.04	-	3.22
Oxygen System Exhaust Gas Composition as % Oxygen	-	46	55	-	51
Overall % of Feed Oxygen Utilized	-	95.5	92.7	-	91.4
Lbs. Oxygen Utilized/Lb. BOD Consumed	-	0.94	0.96	-	0.72
Total Power Required for Liquid Mixing (HP)	-	26.0	11.1	-	11.7
Total Power Required for Gas Recirculation (HP)	-	2.6	9.2	-	10.6
Overall Power Utilized as HP per 1000 Gal. Mixed-liquor	-	0.088	0.125	-	0.137
Total Air Blower Power Utilized (HP)	141	-	-	138.5	-

14j

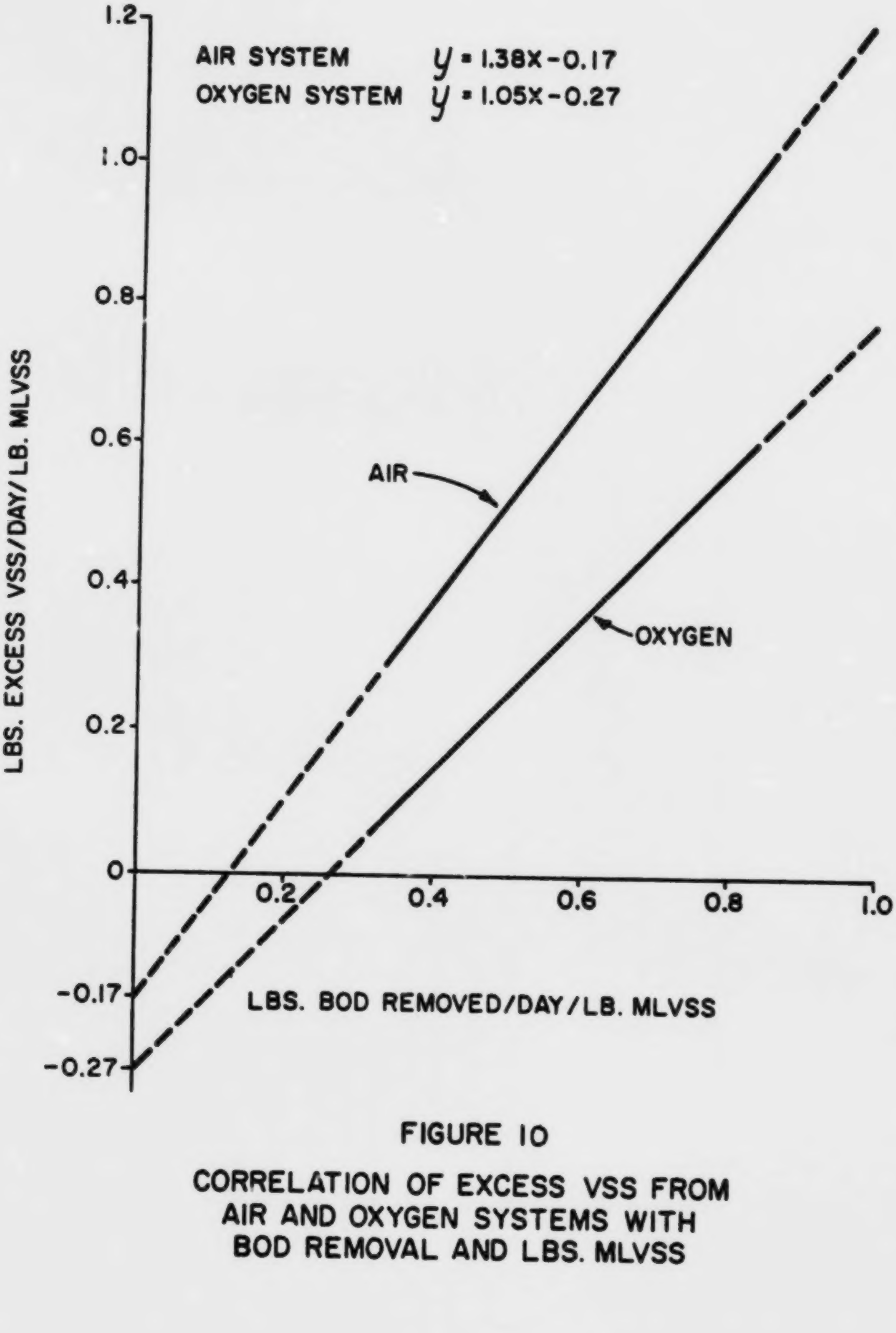


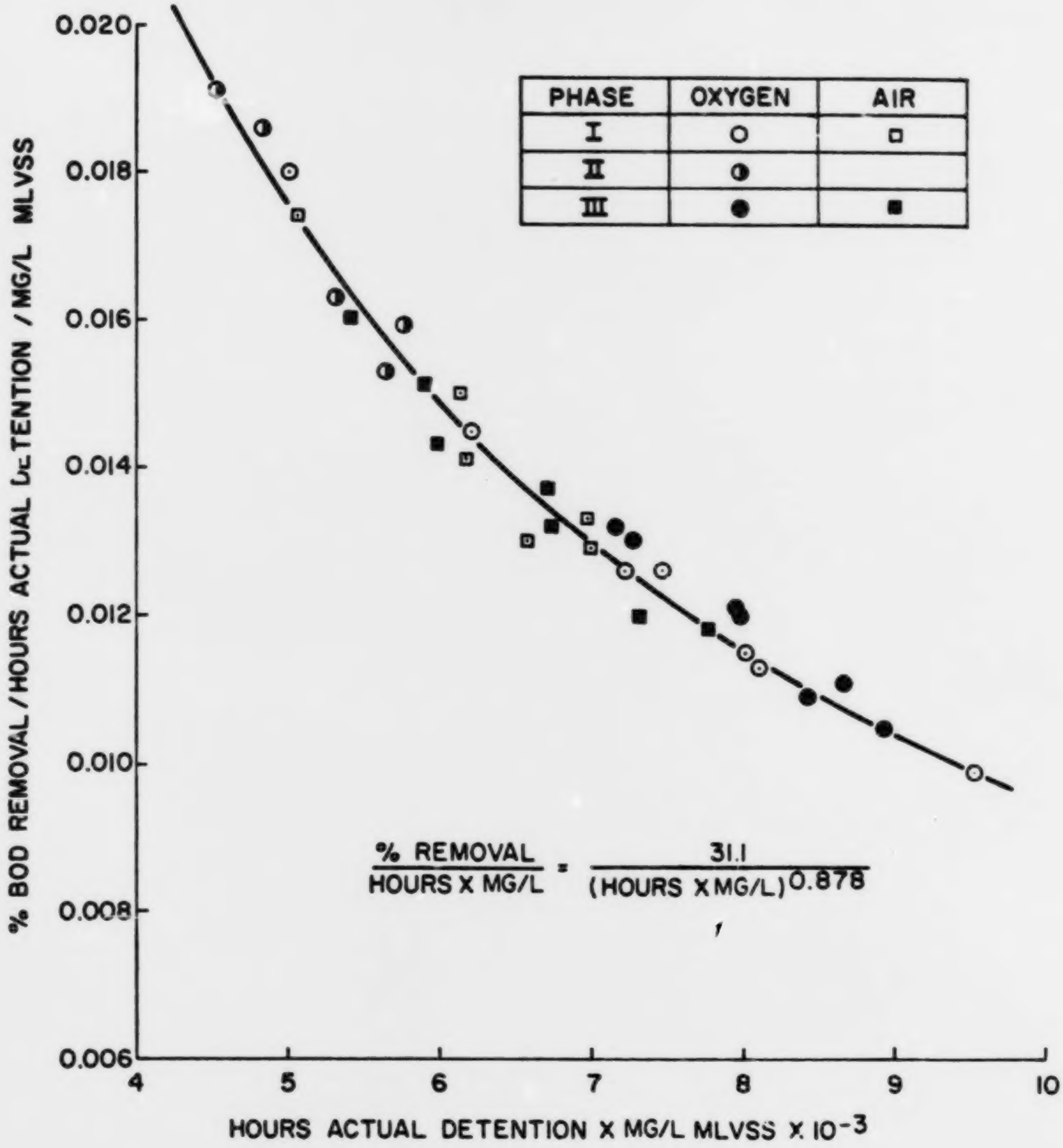
FIGURE 10

CORRELATION OF EXCESS VSS FROM AIR AND OXYGEN SYSTEMS WITH BOD REMOVAL AND LBS. MLVSS

14-k

CORRELATION OF % BOD REMOVAL
AT BATAVIA WITH ACTUAL DETENTION
PERIOD AND MG/L MLVSS

FIGURE 11



14-2

FIGURE 12

SUMMARY OF AERATION RESIDENCE TIMES AND BOD REMOVALS FOR AIR AND OXYGENATION SYSTEM ECONOMIC COMPARISONS

Air System Aeration Detention Time-Hrs. (based on Raw Flow)	Percent BOD Removal Secondary System	Oxygen System Aeration Detention Time-Hrs. (based on Raw Flow)
3.0	84	1.00
4.0	87	1.33
5.0	90	1.66
6.0	92	2.00

14-m

FIGURE 13.

SUMMARY OF VOLUMETRIC ORGANIC LOADINGS AND FOOD TO BIOMASS RATIOS FOR AIR AND OXYGENATION SYSTEM DESIGNS

Aeration Detention Time-Hrs(based on raw flow)	MLVSS Conc. mg/l	Volumetric Organic Loading (lbs BOD/Day 1000 ft^3)	Food to Biomass Ratio (lb BOD/Day lb MLVSS)
		OXYGENATION SYSTEM	
1.00	4420	194	.70
1.33	4420	146	.53
1.66	4420	117	.42
2.00	4420	97	.35
		AIR SYSTEM	
3.0	1625	65	.64
4.0	1625	49	.48
5.0	1625	39	.38
6.0	1625	32	.32

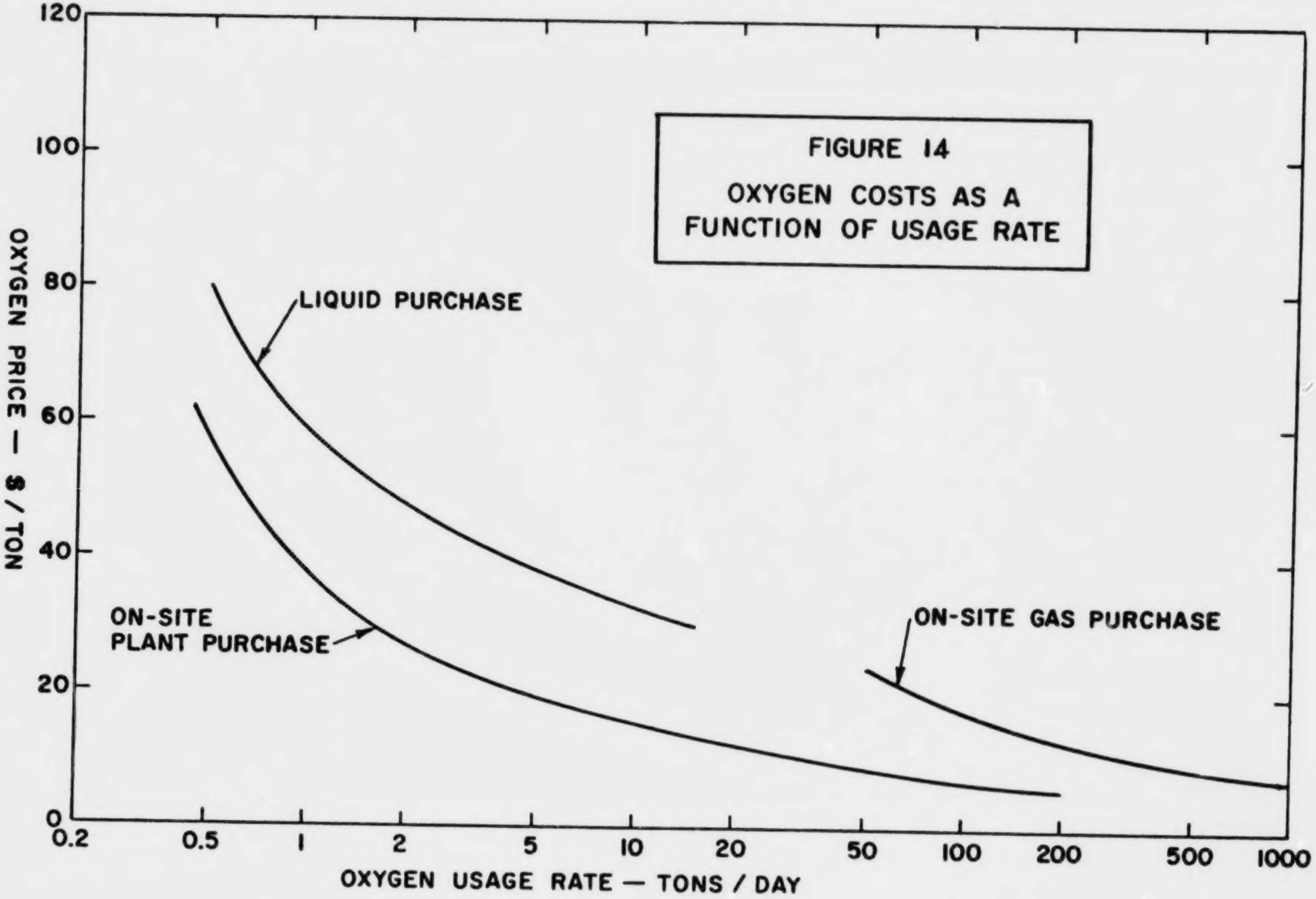


FIGURE 14

OXYGEN COSTS AS A FUNCTION OF USAGE RATE

14-m

FIGURE 15

OXYGENATION SYSTEM POWER REQUIREMENTS

MGD Treated	*Hours Detention	Oxygen Dissolution HP	Oxygen Generation HP	Total Oxygen System HP
1	1.00	6	11	17
	1.33	7	12	19
	1.66	8	14	22
	2.00	10	15	25
6	1.00	37	63	100
	1.33	43	70	113
	1.66	50	78	128
	2.00	57	85	142
30	1.00	180	300	480
	1.33	210	350	560
	1.66	250	385	635
	2.00	280	420	700
100	1.00	510	800	1310
	1.33	620	900	1520
	1.66	740	1000	1740
	2.00	870	1090	1960

* Based on Raw Sewage Flow

FIGURE 16

DIFFUSED AIR AERATION POWER REQUIREMENTS

	BLOWER HP REQUIRED		
MGD TREATED	0.8 CF (NTP) OF AIR PER GAL. OF SEWAGE	1.6 CF (NTP) OF AIR PER GAL. OF SEWAGE	2.4 CF (NTP) OF AIR PER GAL. OF SEWAGE
1	25	50	75
6	150	295	440
30	735	1470	2200
100	2450	4880	7350

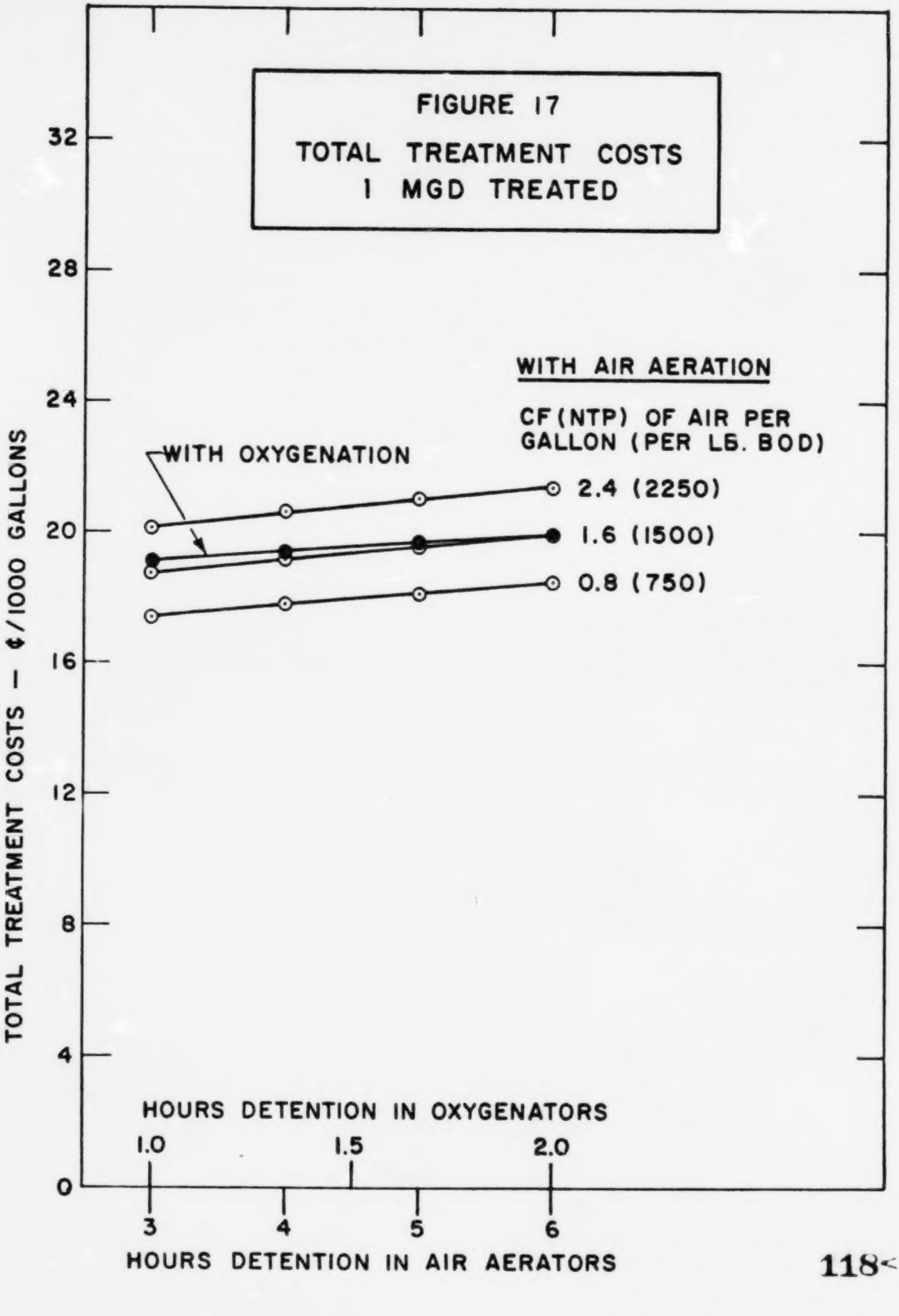


FIGURE 17

TOTAL TREATMENT COSTS
1 MGD TREATED

14-r

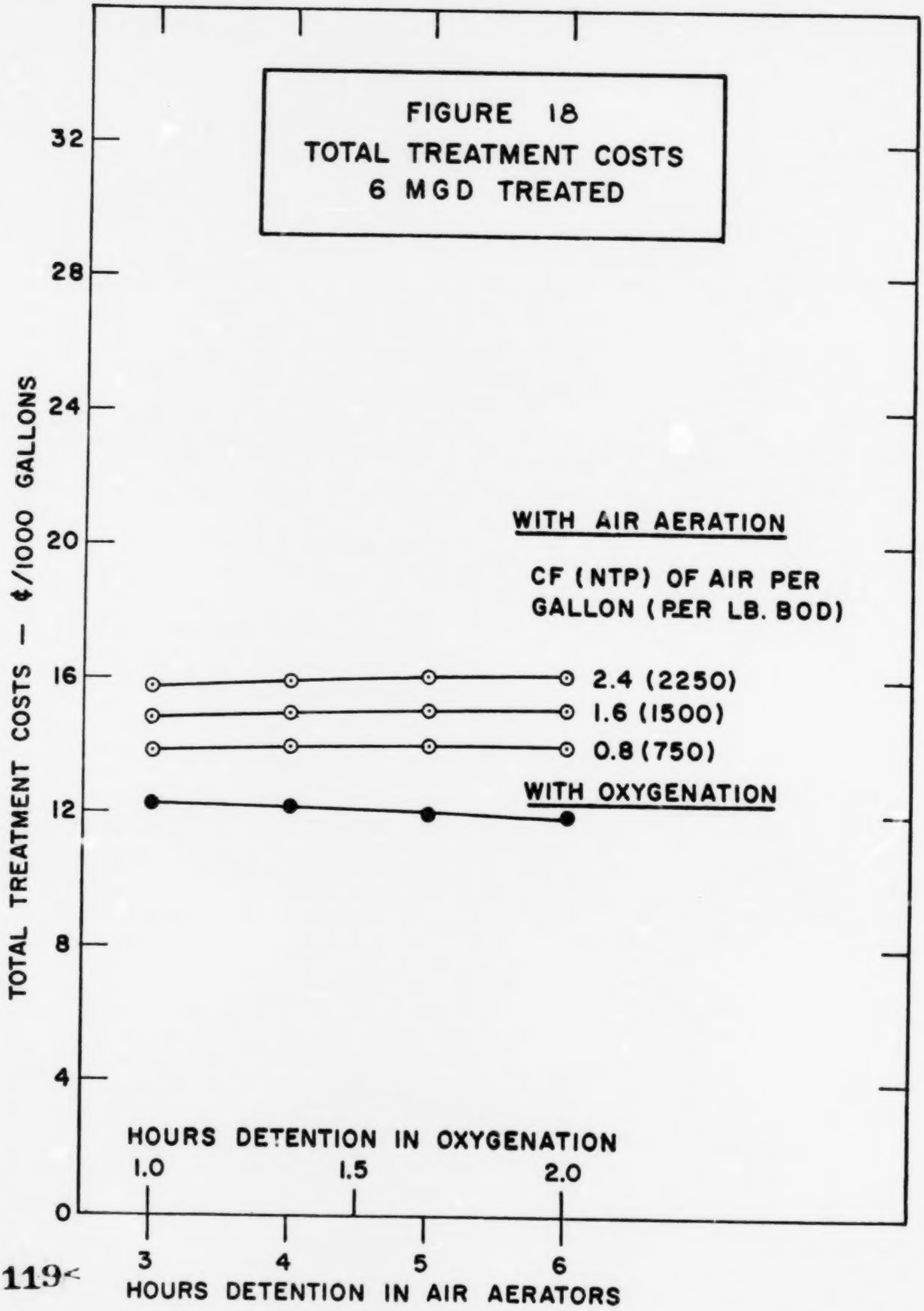


FIGURE 18
TOTAL TREATMENT COSTS
6 MGD TREATED

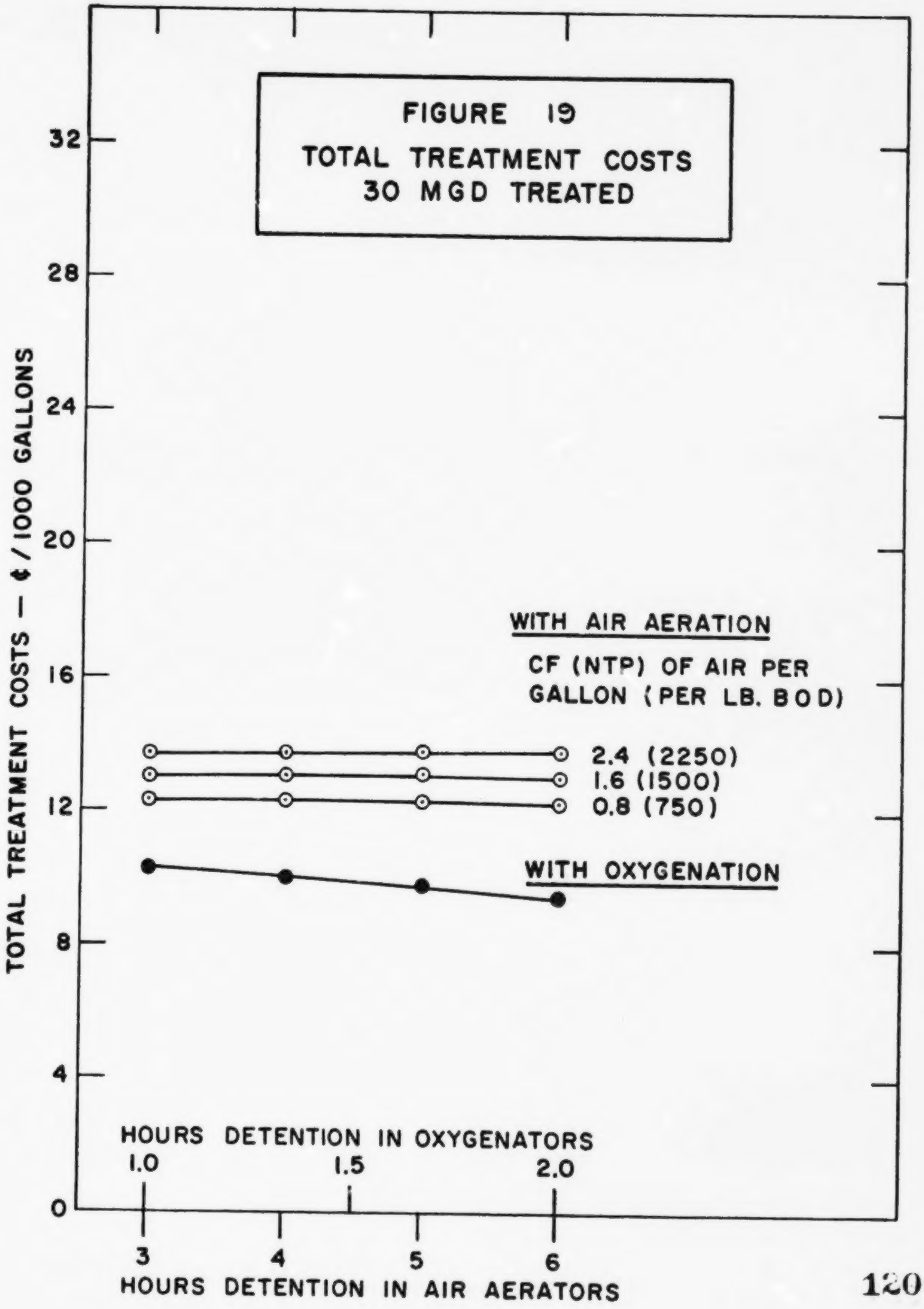


FIGURE 19

TOTAL TREATMENT COSTS

30 MGD TREATED

14-t

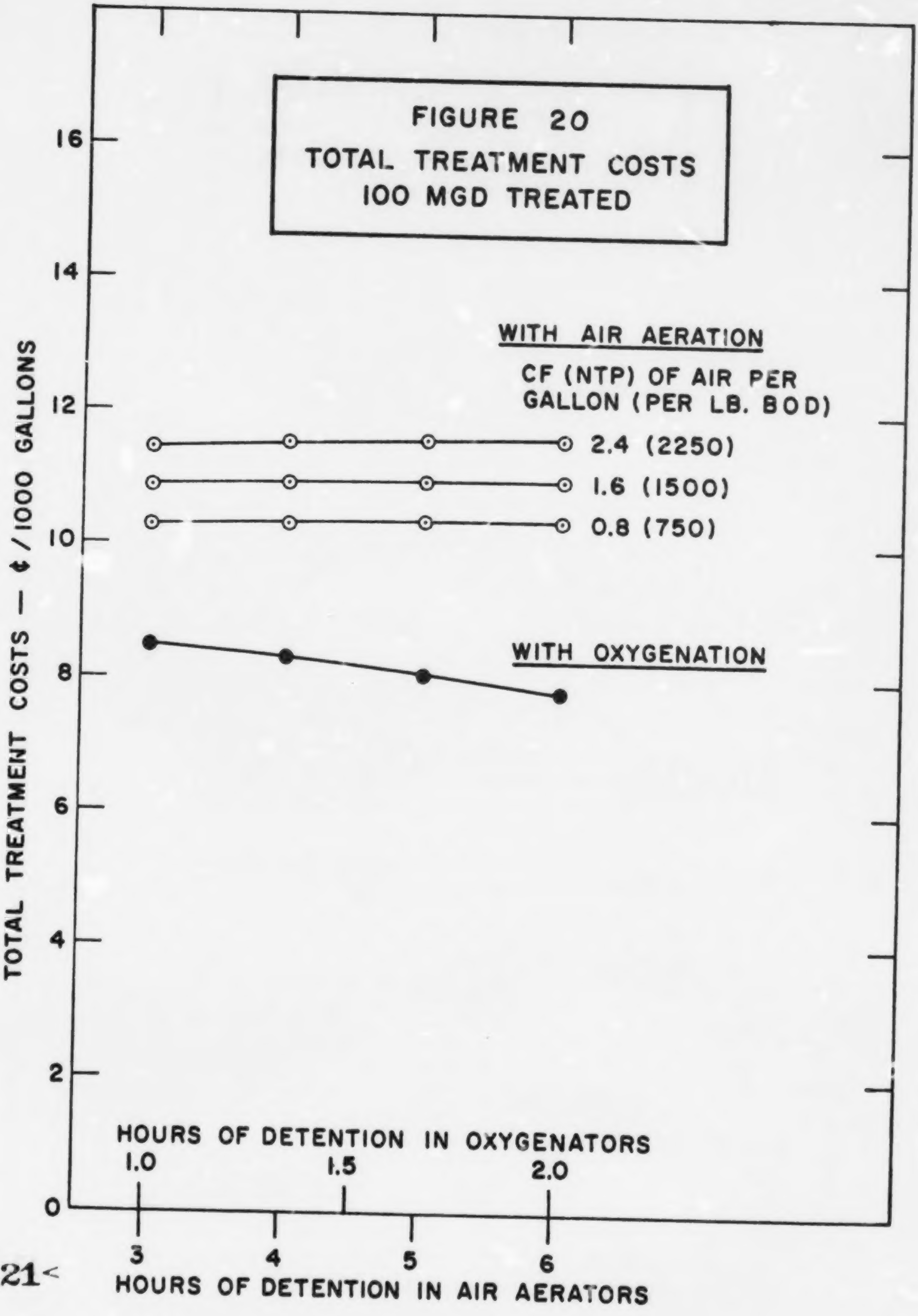


FIGURE 20
TOTAL TREATMENT COSTS
100 MGD TREATED
TOTAL TREATMENT COSTS — ¢/1000 GALLONS
0
2
4
6
8
10
12
14
16
WITH AIR AERATION
CF (NTP) OF AIR PER GALLON (PER LB. BOD)
2.4 (2250)
1.6 (1500)
0.8 (750)
WITH OXYGENATION
HOURS OF DETENTION IN OXYGENATORS
1.0
1.5
2.0
3
4
5
6
HOURS OF DETENTION IN AIR AERATORS

SOLIDS REMOVAL PROCESSES

Sidney A. Hannah

If one were to rank wastewater treatment operations in order of overall importance, separation of solids from liquids would probably assume major importance. This operation may be required to remove objectionable components from a waste, to render the waste more amenable to subsequent treatment or disposal, or to support another treatment operation.

SEDIMENTATION

Sedimentation tanks are employed for various applications which have one common objective - the removal of solid matter from a flowing liquid. The solid matter may have been present in direct suspension, such as in municipal sewage, or may alternatively be a precipitate resulting from prior chemical treatment, such as lime precipitation or alum or iron coagulation.

Horizontal Flow Design

The principal basis of the earliest designs of horizontal-flow sedimentation tank was the "retention time". The objective was to move water so slowly through the tank that there would be ample time for settlement. The nominal detention period allowed was usually more than four hours. However, each drop of water did not take the same time to travel from inlet to outlet, since some degree of "short-circuiting" was bound to occur. In fact, a typical pattern of flow under ideal conditions in an elementary design of a horizontal-flow tank is represented by Fig. 1. Diagram (a), solid line, represents the most favored flow-path, while the dotted lines indicate typical eddies which are induced in the remainder of the tank volume. Diagrams (b) and (c) represent the flows which result, respectively, to entering water being warmer or colder than the water in the tank.

The basic design which has been most widely used for sewage treatment is a tank circular in plan. The inflow is introduced at the center of the tank and the outflow is collected at a peripheral launder. As the general direction of flow is mainly a radial spread from the center, it is reasonable to regard this design as in the horizontal-flow category.

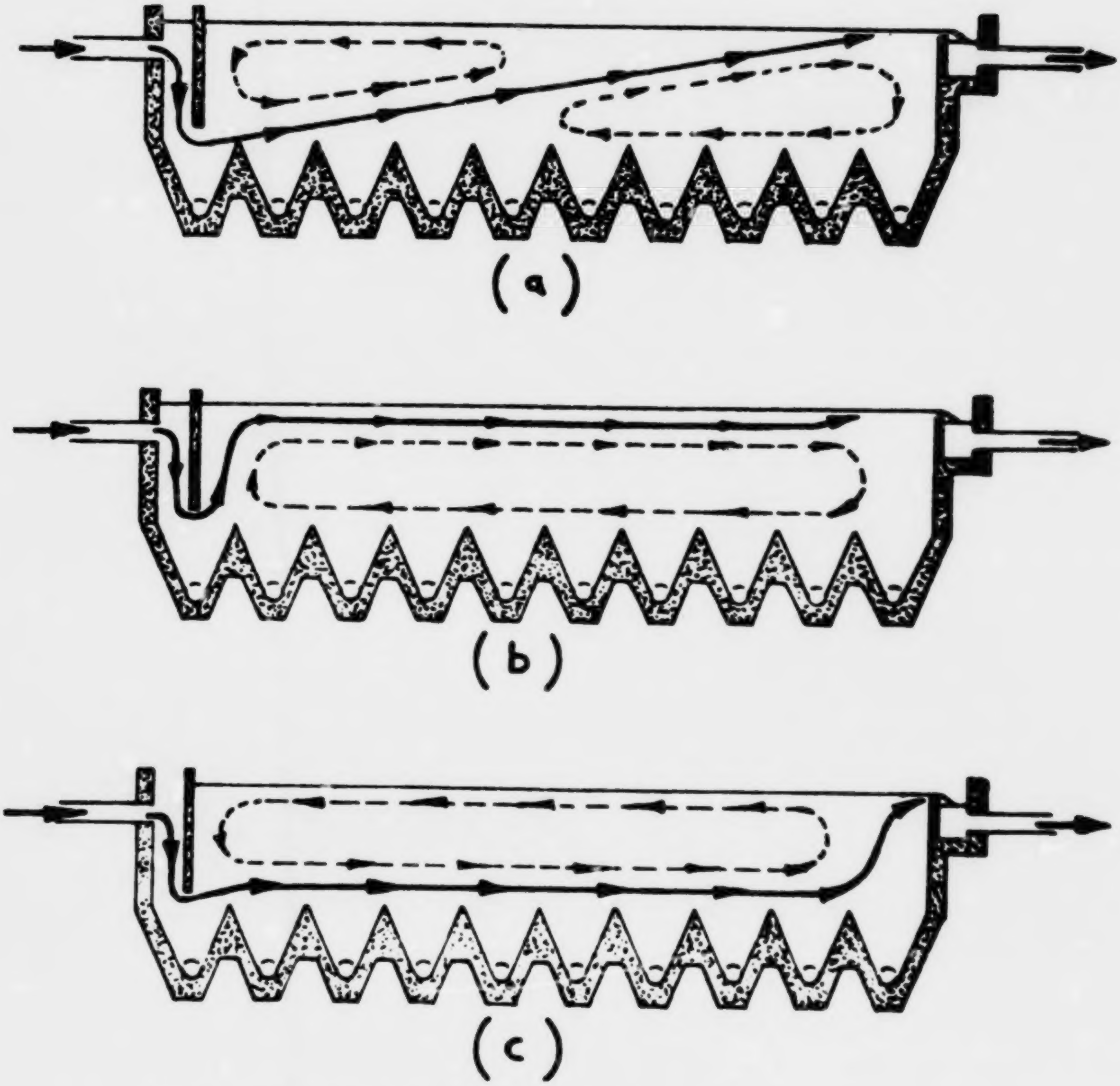


Figure 1—Sedimentation tank of horizontal-flow type under different conditions of relative temperature.

Vertical-Upflow Design

The improved performance obtained in upflow tanks has led to a variety of new designs, incorporating variants of the upflow principle. These new designs are designated by various proprietary names - Accelator, Clarifow, Flocsettler, Reactor-Clarifier, etc. One of the earliest of the upflow tanks was the Spaulding Precipitator, shown in Fig. 2. In this design, the flow is introduced into the center of the tank and flows upward through a blanket of previously formed solids.

The principal advantages of the upflow versus the horizontal tanks are (1) improved flow control, and (2) sludge blanket effect. Salt-injection tests performed on a variety of tank designs clearly show the superior efficiency of the upflow principle in the matter of flow control. The radial-flow still largely used in sewage treatment is distinctly inferior, being subject to short-circuiting.

Tube and Lamella Settlers

In an ideal settling basin, as defined by Camp, the paths of all discrete particles will be straight lines, and all particles with the same settling velocity will move in parallel paths. The settling pattern shown in Fig. 3 would be the same for all longitudinal sections. It is apparent from this that as the interval (h) is reduced, the size of the basin required to remove a given percentage of the incoming settleable material decreases. Many devices have been proposed since this principle was first proposed by Hazen in 1904 and further developed by Camp in 1946. None of the proposed devices were accepted commercially until the recent introduction of the device called the tube settler. These consist essentially of closely packed small diameter tubes, 1-4 inches in diameter and 2-4 feet in length, inclined at some angle to provide for removal of sludge as the water flows upward through the tubes. Detention times are in the order of 6 minutes and less. The tubes provide as much as 24 hours of sludge storage, depending of course, on the amount of suspended solids, and sludge is readily removed by gravity drainage. A schematic diagram of the tube settler integrated into a complete clarification device which consists of coagulation-flocculation, a tube settler unit and a mixed media filter is shown in Fig. 4. More recently, a device consisting of parallel plates rather than tubes has become available. The device is called a lamella. Flow is co-current in contrast to the countercurrent flow of the tube settlers. The plate arrangement in a tank is shown in Fig. 5.

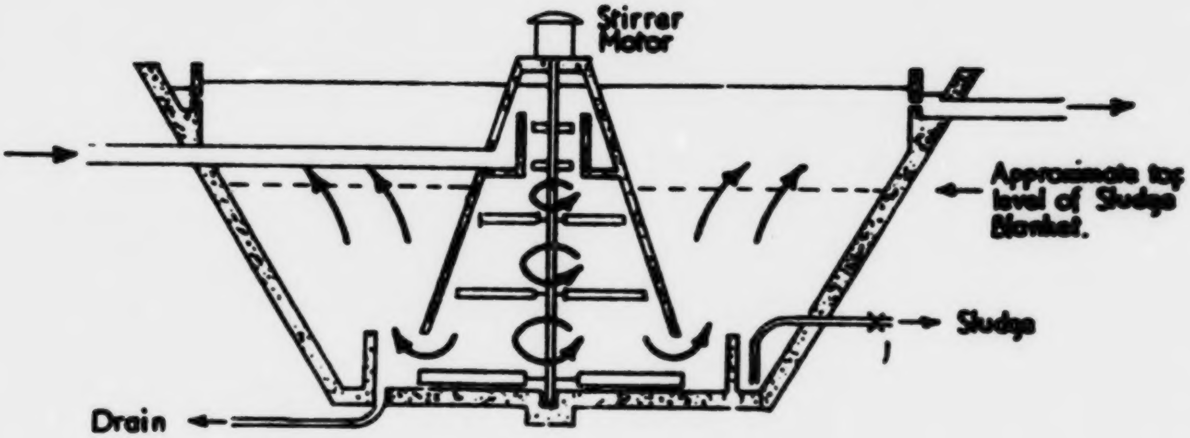

Stirrer Motor
Approximate top level of Sludge Blanket.
Sludge
Drain

FIG. 2 — Spaulding Precipitator

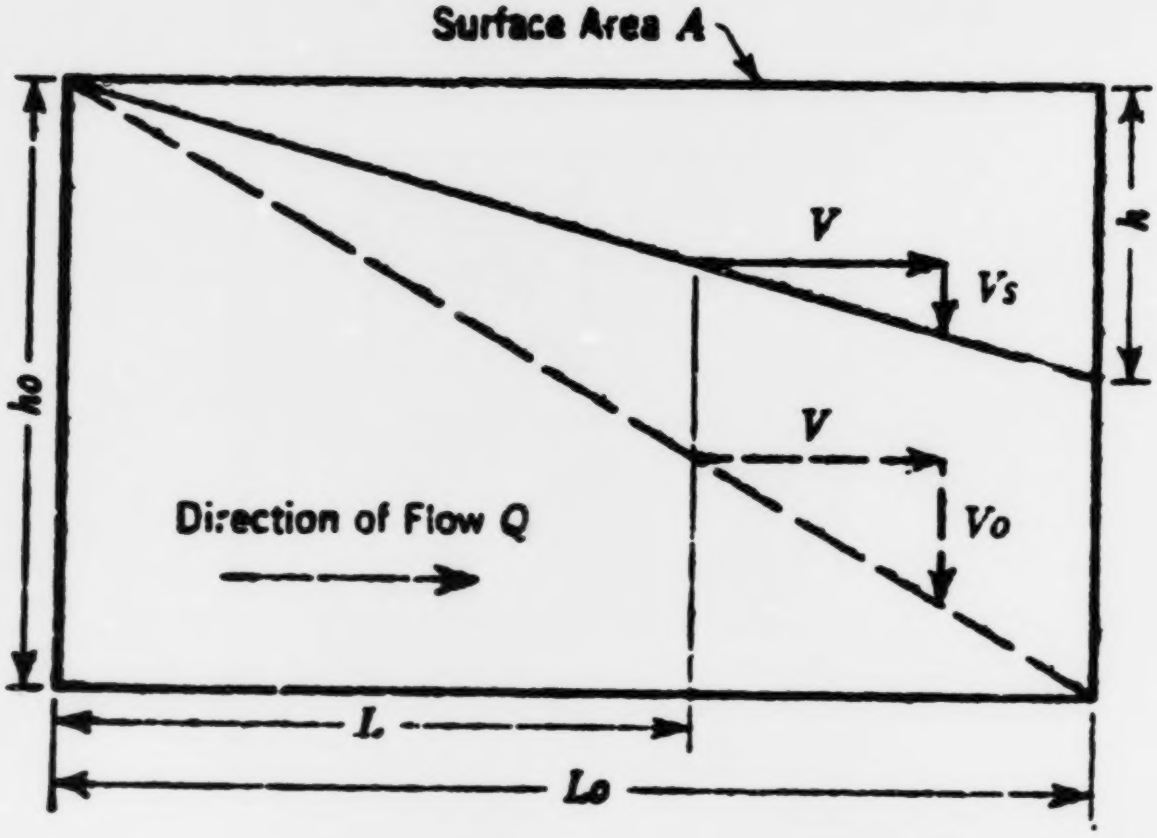

Surface Area A
V
Vs
h
ho
V
Vo
Direction of Flow Q
L
Lo

FIG. 3 **Idealized Settling Paths of Discrete Particles in a Horizontal Flow Tank**

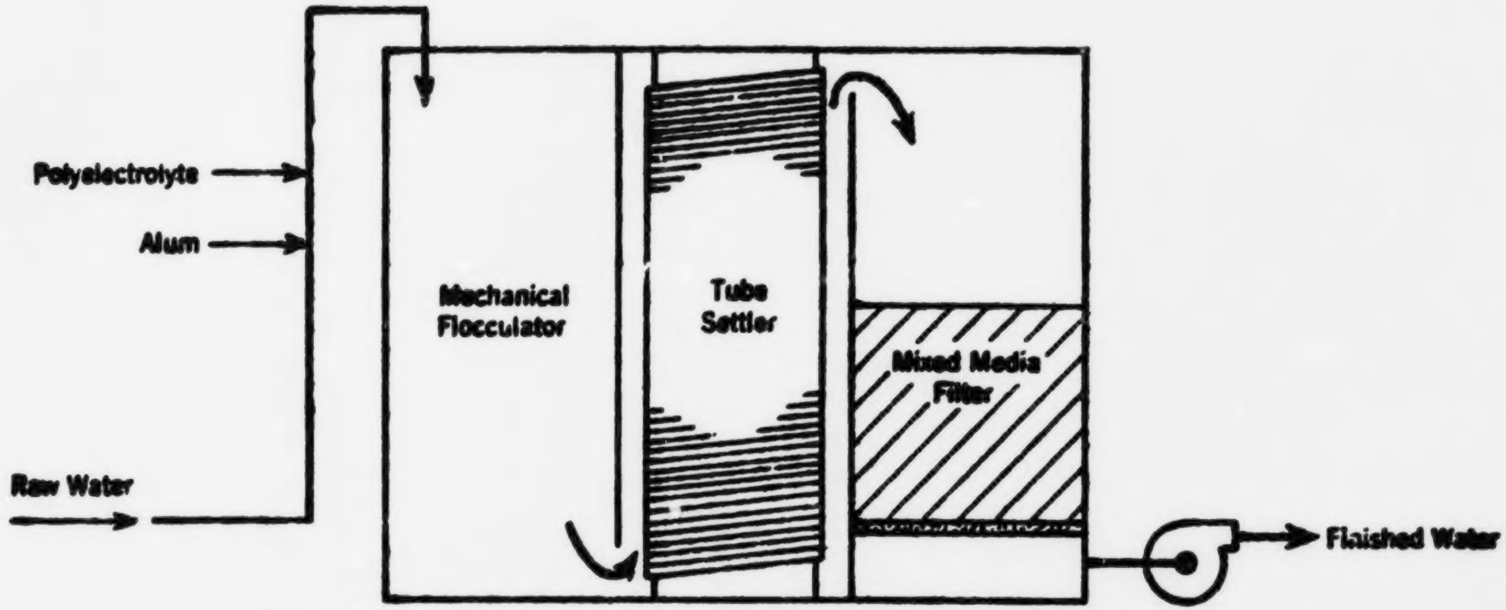

Polyelectrolyte
Alum
Raw Water
Mechanical Flocculator
Tube Settler
Mixed Media Filter
Finished Water

Fig. 4. Schematic Diagram of Apparatus Used in Field Tests of Tube Settler

Reproduced from best available copy.

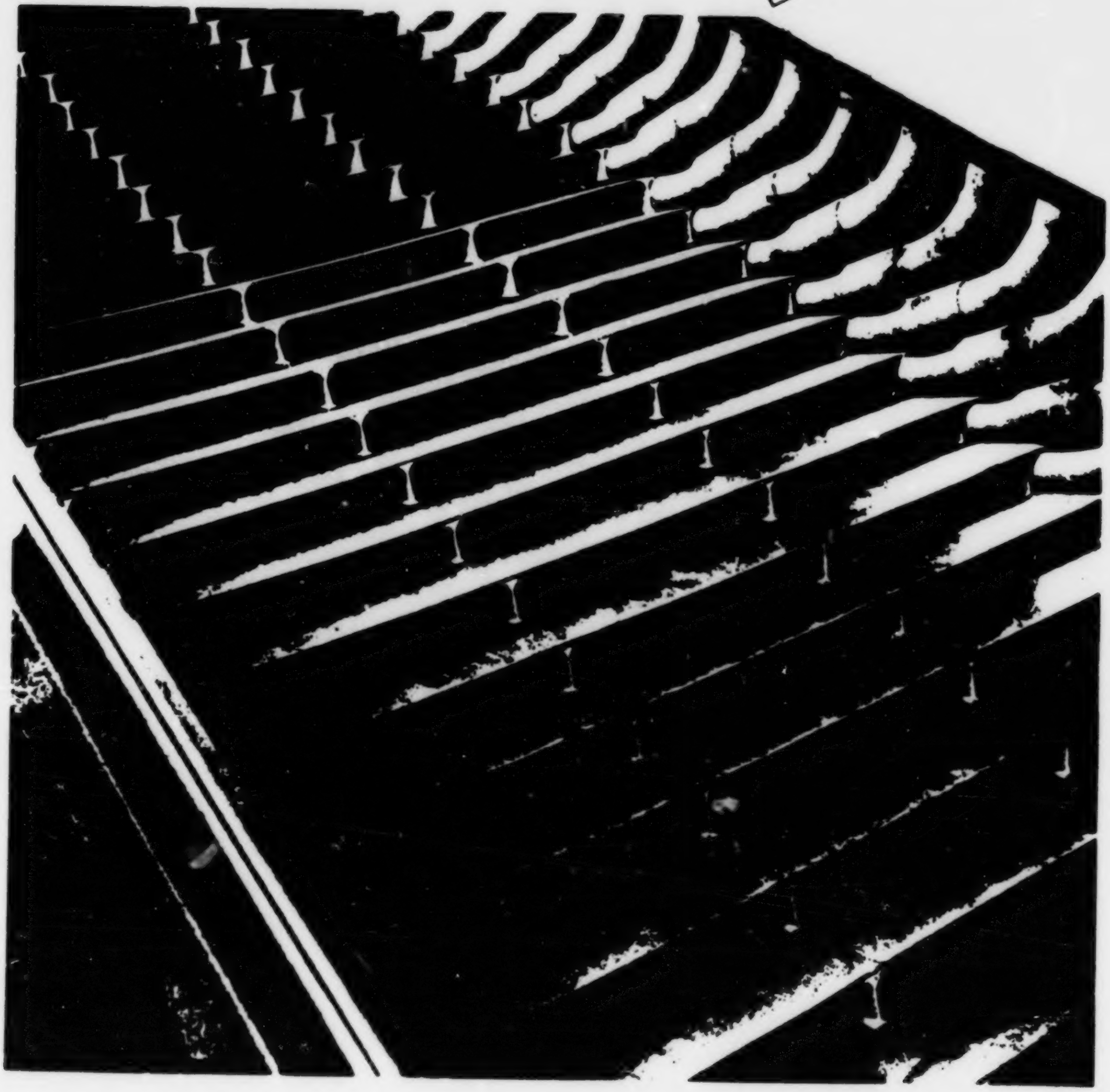


Figure 5 Lamella Separator

DISSOLVED AIR FLOTATION

The use of dissolved air to float suspended solids was first used in industrial operations. In recent years, the process has been adapted to domestic wastewater and particularly to sludge thickening. The process achieves the separation of suspended particles by attachment of gas bubbles to the suspended particles, thereby reducing the effective specific gravity of the particles to less than that of the water.

A. A flotation system using the pressurization and de-pressurization sequence consists of the following elements (Fig. 6).

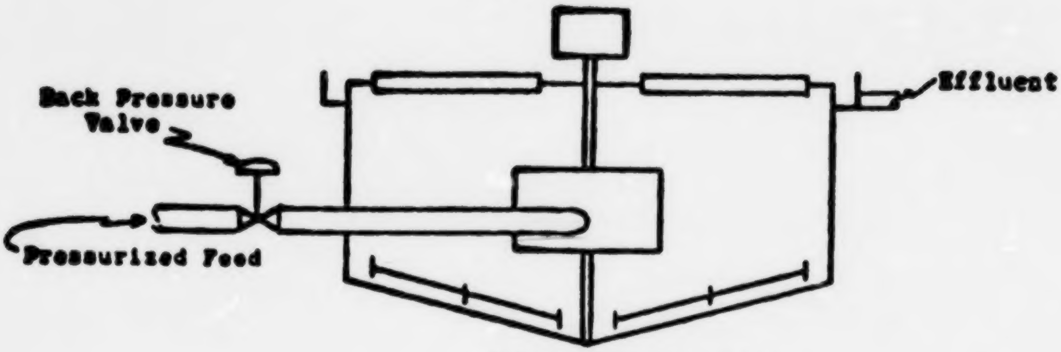
1. Pressurizing pump
2. Air injection facilities
3. Retention tank or contact vessel
4. Back-pressure regulating device
5. Flotation device
6. Facility for addition of chemicals if needed

B. Advantages and Disadvantages

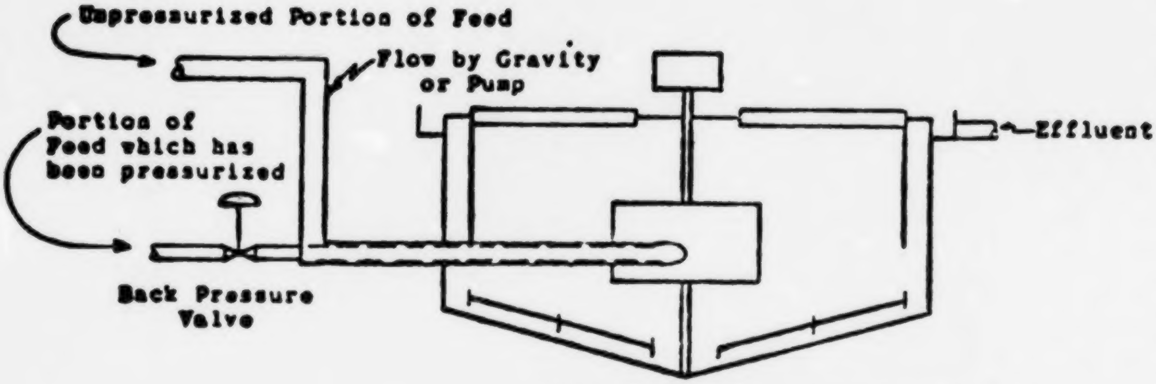
1. Much reduced retention time - 10-20 minutes
2. Greater solids concentration in float than in settled sludge
3. Greater efficiency of solids recovery
4. Offers mechanical control over the process
5. Increased cost of operation for pumping, etc.
6. Need to remove top and bottom solids

SCREENING DEVICES

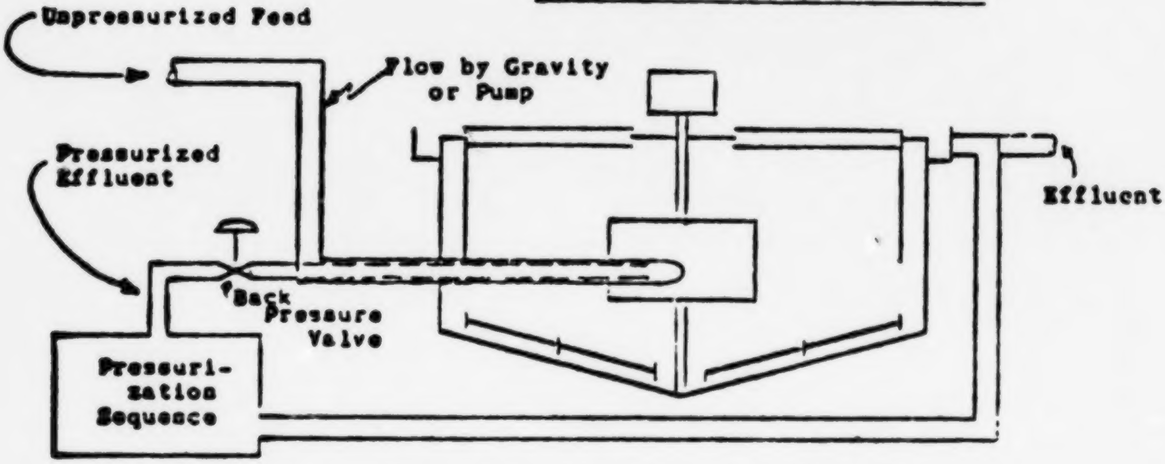
Microscreening is a form of simple filtration by straining (Fig. 7). These mechanical filters consist of a rotary drum which revolves on a horizontal axis. The peripheral surface of the drum is covered with a stainless steel fabric. The effectiveness of the woven mesh screen for retaining fine particles is dependent on the size of the openings in the screen and on the pattern of the weave. Influent enters the open end of the drum and is filtered through the fabric with the intercepted solids being retained on the inside surface of the fabric. As the drum rotates, the solids are transported and continuously removed at the top of the drum by pumping strained effluent, under pressure, through a series of spray nozzles which extend the length of the drum. The solids and wash water are collected in a central trough within the drum and discharged through a hollow axle. The microstraining device is available in several unit sizes ranging from 5' in diameter and 1' width with a capacity of 0.05 to 0.5 mgd to 10' diameter and 10' wide with a capacity of 3-10 mgd.


Back Pressure Valve
Effluent
Pressurized Feed

Total Pressurization


Unpressurized Portion of Feed
Flow by Gravity or Pump
Portion of Feed which has been pressurized
Effluent
Back Pressure Valve

Partial Pressurization of Feed


Unpressurized Feed
Flow by Gravity or Pump
Pressurized Effluent
Effluent
Back Pressure Valve
Pressurization Sequence

Partial Pressurization of Effluent

Methods employed for partial and total pressurizations.

Figure 6

FIGURE 7

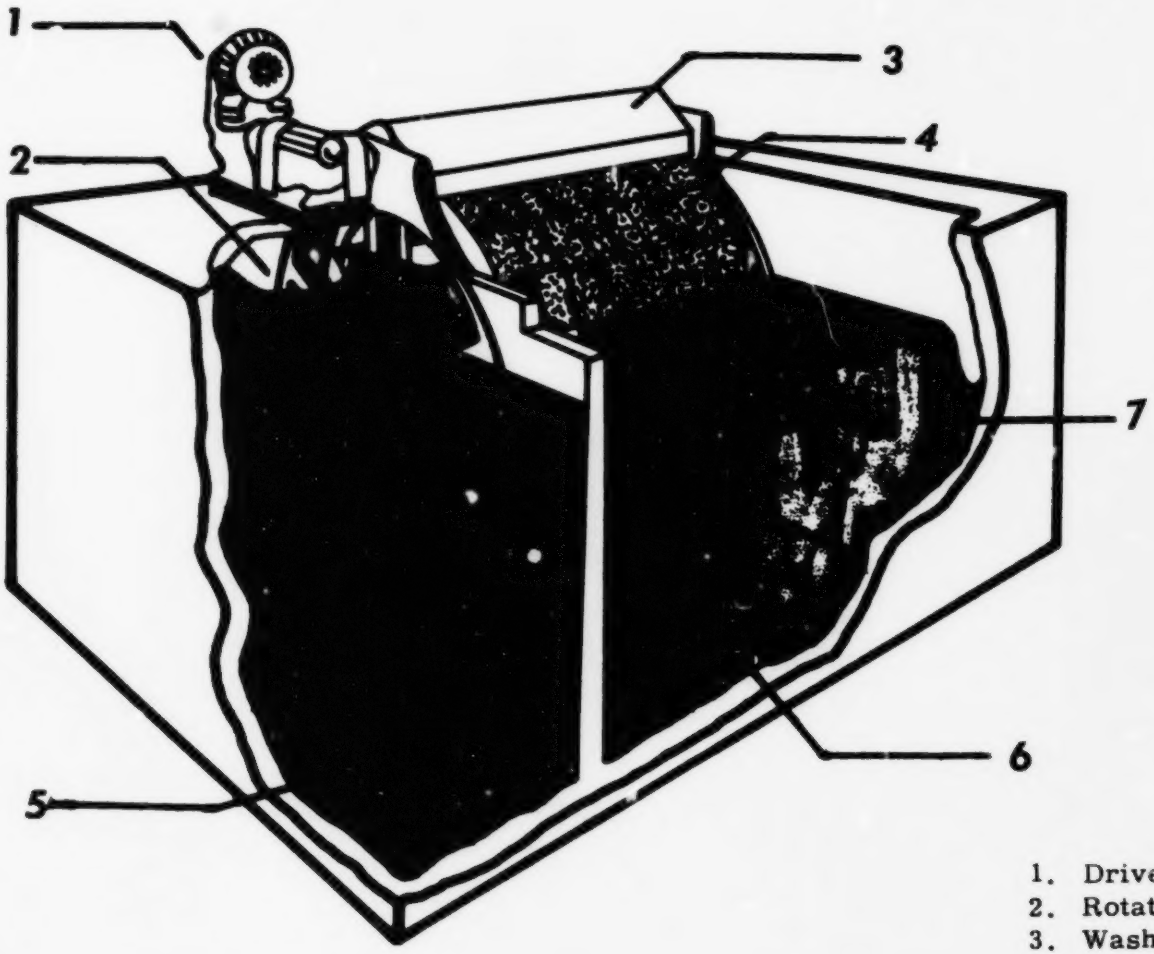


Fig. 1 Typical Microstrainer Unit

1. Drive Unit
2. Rotating Drum
3. Wash-water Jets
4. Micro-fabric
5. Influent Chamber
6. Effluent Chamber
7. Effluent Weir

Not Shown

Wastewater Hopper
Ultra-violet Lamp
Wash-water Pump

Microscreening devices have found their greatest application to treatment of river waters, and information on their performance on wastewaters is quite scarce. The effect of aperture size on removal efficiencies and flow rate is shown below.

Fabric	Removal Efficiency		Flow
	Solids	BOD	gals/hr/ft^2
Mark 0 (23 microns)	70-80%	60-70%	400
Mark 1 (35 microns)	50-60%	40-50%	600

The advantages of microstraining are the low initial capital cost and ease of operation. The disadvantages are the incomplete solids removal and the inability to handle solids fluctuations.

Capital and Operating Costs

The capital and operating costs associated with microstraining have been prepared by Smith and are shown in Fig. 8. For a 10 mgd plant, total cost for microstraining is calculated to cost about 1.5¢/1000 gallon.

In-Depth Filtration

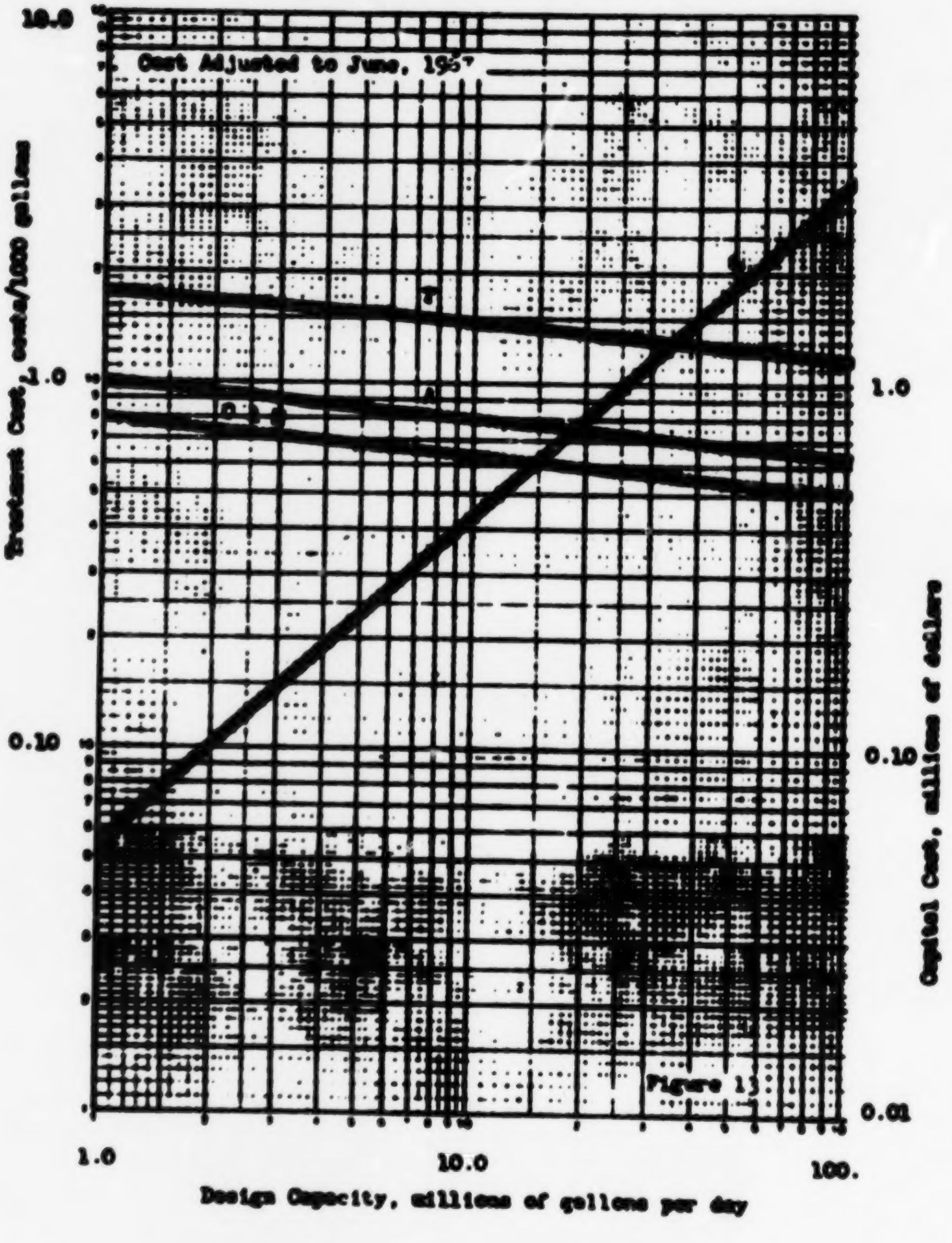
In-depth filtration is the passage of a fluid through a bed of granular media designed to permit the captured particles to be retained within the filter. The degree of penetration and solids removal efficiency can be altered by changing the size and character of the particulates to be removed as well as the size and composition of the filter media itself.

A basic prerequisite for the operation of rapid sand filters is that good coagulation and flocculation must be obtained. Barring adequate pretreatment of the wastewater, filtration efficiency is decreased as evidenced by "breakthrough" of floc. With good pretreatment by coagulation, higher filtration rates are attainable while still maintaining clarity of the effluent.

It is beyond the scope of this talk to discuss the subject of coagulation and flocculation. A variety of coagulants and flocculants are available including various salts of aluminum and iron, lime and organic polymeric flocculants. The former inorganic salts, in the proper dosages can also provide for precipitation of phosphate in addition to clarification.

MICROSTRAINING OF SECONDARY EFFLUENT

Capital Cost, Operating & Maintenance Cost, Debt Service
vs.
Design Capacity



C - Capital Cost, millions of dollars
A - Debt Service, cents per 1000 gallons(4 1/2% - 25 yr.)
O & M - Operating and Maintenance Cost, cents per 1000 gallons
T - Total Treatment Cost, cents per 1000 gallons

FIG. 8

In the past 70 or 80 years there has been a gradual improvement in the basic process of media filtration. Some of the earlier work centered on increasing the filtration rates of slow sand filters to the modern rapid sand filter rates. Various cleaning methods were also developed including backwashing and surface scouring. More recently, the engineering advances have been concerned with modifications of filter media that would allow greater production of high quality water from a given filter area.

The evolution of filter design is illustrated in Fig. 9. The cross-section shown at the top represents the rapid-sand filter which is in common use in many filter plants today. Typical effective size of the media used is 0.5 mm although effective sizes from 0.35 to 1.0 mm have been used. During filter backwashing, the sand grades hydraulically with the finest particles rising to the top of the bed. As a result, most of the material removed by the filter is removed at very near the surface of the bed. Only a small part of the total voids in the bed are used to store particulates and headloss increases very rapidly. When secondary effluent is being processed, the high solids concentration will blind the surface in a very short time. As much as 75-95% of the headloss, under these conditions, will occur at the upper 1-inch layer of the filter. Filter runs will be so short as to be prohibitive. Further, floc breaking through the topmost layers, have increased opportunity to pass through the entire filter since voids become increasingly larger with increase in depth.

One approach to increasing the effective filter depth is the use of a dual media bed using a discrete layer of coarse coal above a layer of fine sand. The filter provides basically a two-layer effect to achieve increased penetration of particles. The amount of sand is reduced to afford lower headlosses at the higher throughput rates used. Normally, such a bed is designed so that 24 inches of anthracite coal, with a nominal size of about 1 mm, overlays a 6 inch sand layer with a size of about 0.45 mm. Hydraulic stratification still occurs following backwashing but the difference of specific gravity is such that the larger coal remains on top of the sand. The bulk of filtration is accomplished in the upper layers of the coal and at the top inch or two of the sand bed.

With low applied turbidities and constant rate operation, the coal-sand media bed has demonstrated an ability to operate in the range of 4-5 gal/min/sq/ft of filter surface area. A defect in this design is that if a flow change occurs, the particles held in the relatively large void volume of a coal bed, can become dislodged and will be captured by the fine layer of sand and the filtration run would have to be terminated either because of high head loss at the coal-sand interface or because of breakthrough of particles through the relatively shallow sand bed. This design, then, presents a serious inconsistency in design.

Ideally, the effluent should pass through as fine a filter material as is feasible. This ideal design is illustrated in the bottom cross-section of a filter uniformly graded from coarse to fine from top to bottom.

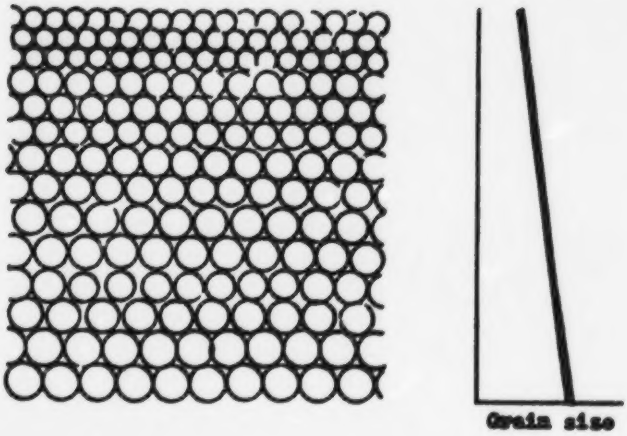


Figure 1
Cross-Section Through
Single-Media Bed
Such as Conventional
Rapid Sand Filter

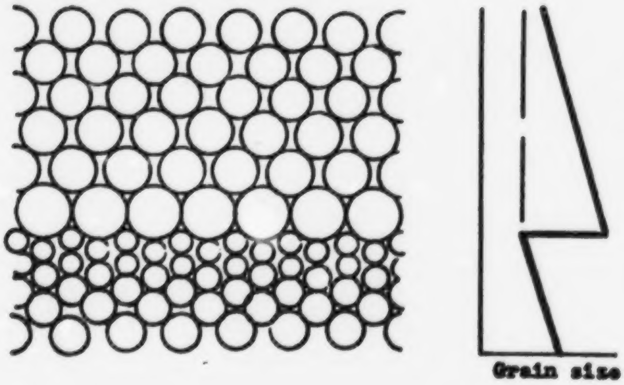


Figure 2
Cross-Section Through
Dual-Media Bed
Coarse Coal Above
Fine Sand

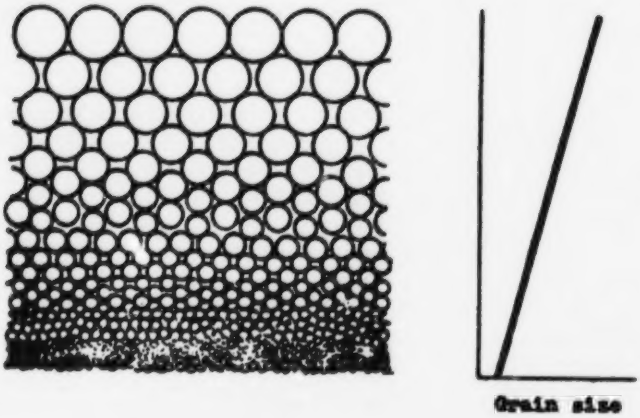


Figure 3
Cross-Section Through
Ideal Filter
Uniformly Graded From
Coarse to Fine
From Top to Bottom

FIG. 9

One solution is the use of three materials of differing specific gravity of such size gradation that some intermixing of the materials occur at the interfaces of the bed layers. The third media material is garnet which has a specific gravity of about 4.2. An ideal filter bed, then, would consist of about 60% of anthracite with a size of 1.0 to 1.5 mm at the top, 30% sand with a size of 0.4 to 0.5 mm at the middle and 10% of garnet with a size of about 0.15 mm. The materials are so sized that intermixing occurs at the interfaces. In this ideal filter, the effluent is passed through increasingly finer media. The uniform decrease in media particle size with filter depth allows the entire filter depth to be used for floc removal and storage.

Cost of the filtration step is shown in Fig. 10. For a 10 mgd plant this cost amounts to about 3.5¢/1000 gallon, when operating the filter at 4 gpm/sq ft. Further economy, of course, can be obtained at higher rates. Rates as high as 6-8 gpm/sq ft have been shown to be feasible when proper pretreatment of coagulation-flocculation followed by sedimentation is practiced.

Moving Bed Filter Technique

A new filtering technique has been evaluated by Johns-Manville under contract with FWQA. The technique is designed to overcome the problem of surface clogging and to achieve what is obtainable by multi-media filtration. A schematic diagram of the moving bed filter is shown in Fig. 11.

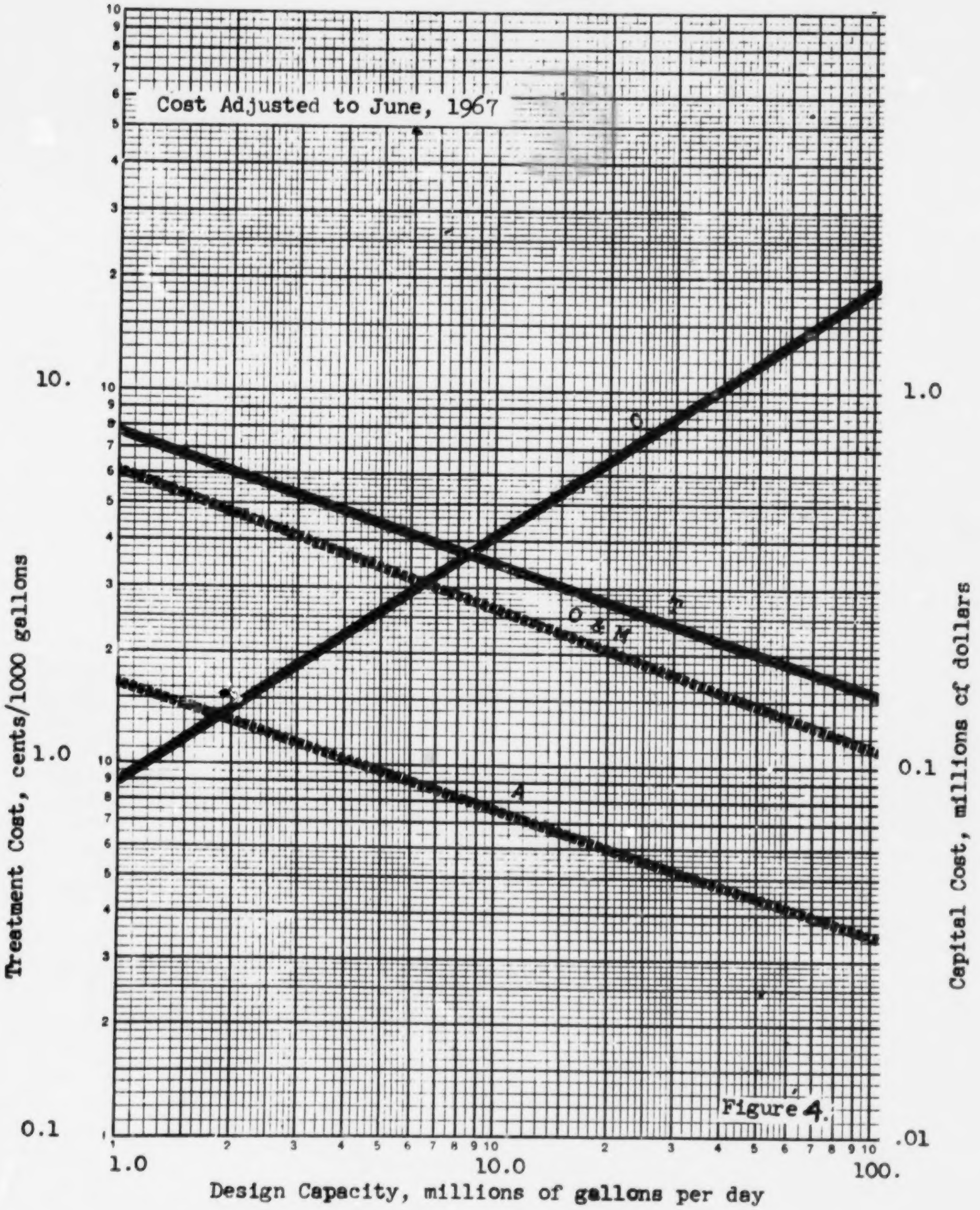
The unit is basically a sand filter. Particulate matter is removed as the water passes through the sand (0.6 to 0.8 mm). As the filter surface becomes clogged, the filter media is moved forward by means of a mechancal diaphragm. The clogged filter surface is removed either hydraulically or, as shown, mechanically thereby exposing a clean filter surface. The sand and accumulated sludge is collected and washed. The sand is returned via a hopper to the base of the bed. The unit is thus a form of countercurrent extraction device feeding sand countercurrent to the water being filtered. The moving bed filter has a renewable filter surface analogous to the microstrainer and the advantage of depth filtration comparable to the coarse media filter. The unit does not have to be taken off-stream for backwashing. In theory, 1% of the filter is being backwashed 100% of the time compared to the conventional practice of backwashing 100% of the filter 1% of the time.

Several pilot MBF units have been built to date and used to treat settled and non-settled trickling filter effluents and primary effluent. The system lends itself well to the use of chemical aids ahead of filtration because of designed flexibility to handle high solids loadings.

Figure 10

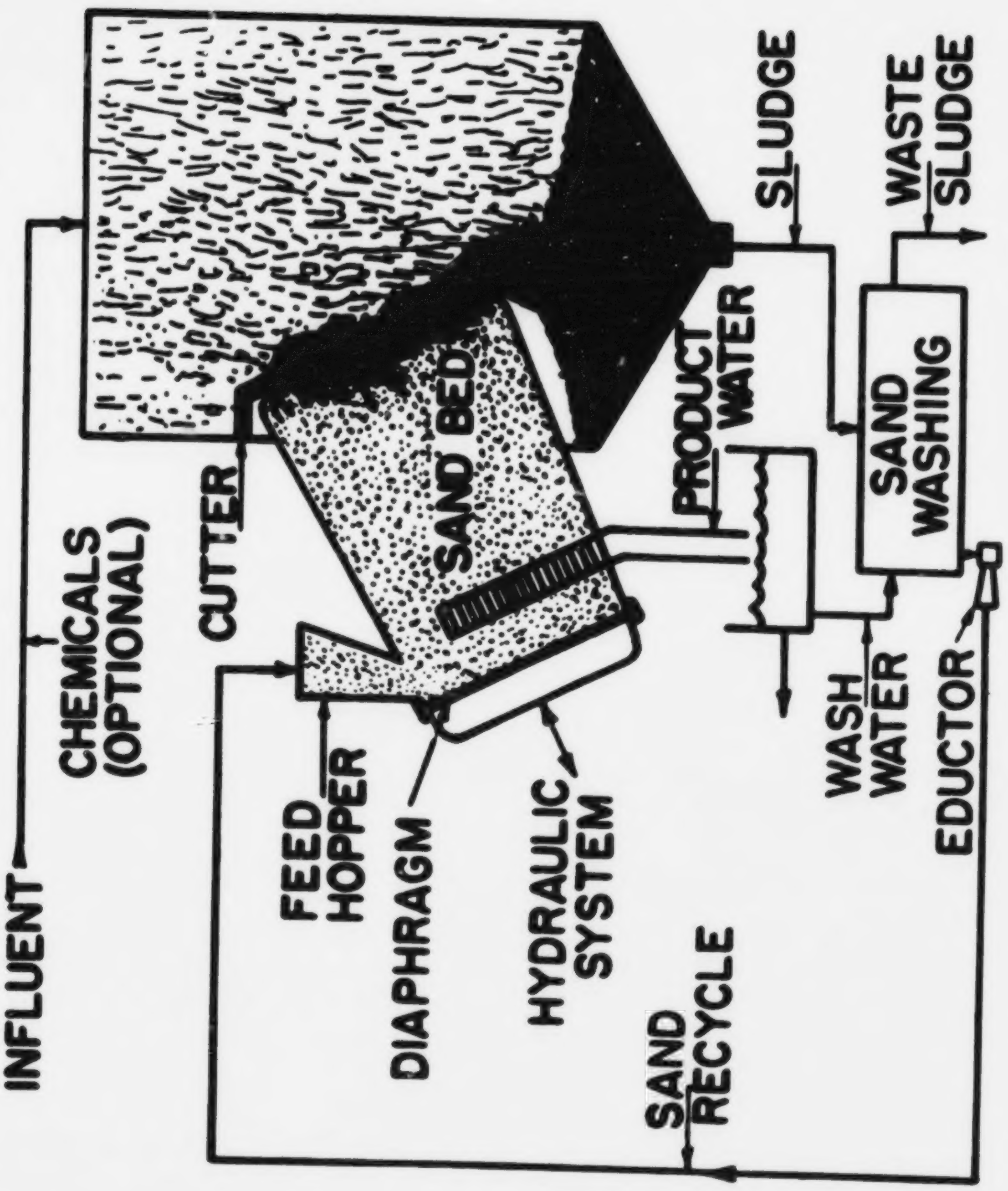
FILTRATION THROUGH SAND OR GRADED MEDIA - 4GPM/SQ FT

Capital Cost, Operating & Maintenance Cost, Debt Service
vs.
Design Capacity



C = Capital Cost, millions of dollars
A = Debt Service, cents per 1000 gallons(4 1/2% - 25 yr.)
O & M = Operating and Maintenance Cost, cents per 1000 gallons
T = Total Treatment Cost, cents per 1000 gallons

FIGURE 11



BASIC CONCEPT OF MOVING BED FILTER

INFLUENT
CHEMICALS (OPTIONAL)
CUTTER
FEED HOPPER
DIAPHRAGM
HYDRAULIC SYSTEM
SAND BED
PRODUCT WATER
SLUDGE
SAND WASHING
WASTE SLUDGE
WASH WATER
EDUCTOR
SAND RECYCLE

Fig. 12 shows some data with the MBF treating non-settled trickling filter effluent with various dosages of alum and coagulant aid. Product quality was maintained at about 10 BOD while being fed an influent with a widely varying BOD content of ≈40 to as high as 180 BOD.

It is too premature to talk about cost of this method of filtration. Design and performance information developed from these pilot units will be used to obtain these costs.

ULTRAFILTRATION

One of the newest unit processes to separate solids from liquids is the operation known as ultrafiltration. While this method has been under development over the past 10 years, it is only within the past two-three years that has seen the commercial development of this process.

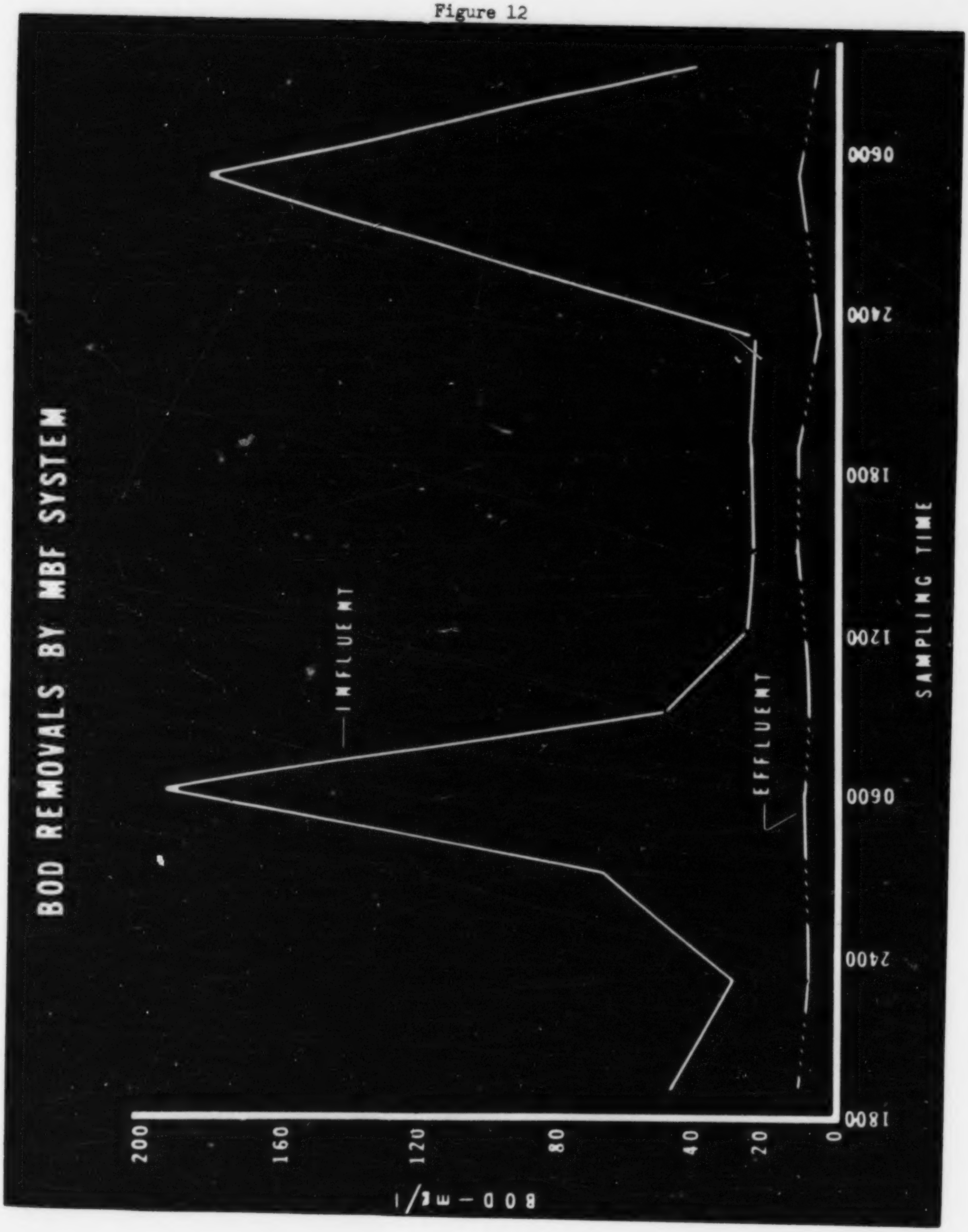
Ultrafiltration is closely related to reverse osmosis with the distinction generally made on the basis of size of particle separated. Reverse osmosis removes all molecular sizes including inorganic salts. Ultrafiltration generally will not separate molecules smaller than ≈500-1000 MW, thus inorganic salts are not separated from solution. Ultrafiltration uses pressures of about 50 psi in contrast to reverse osmosis where pressures in excess of 500 psi are generally used.

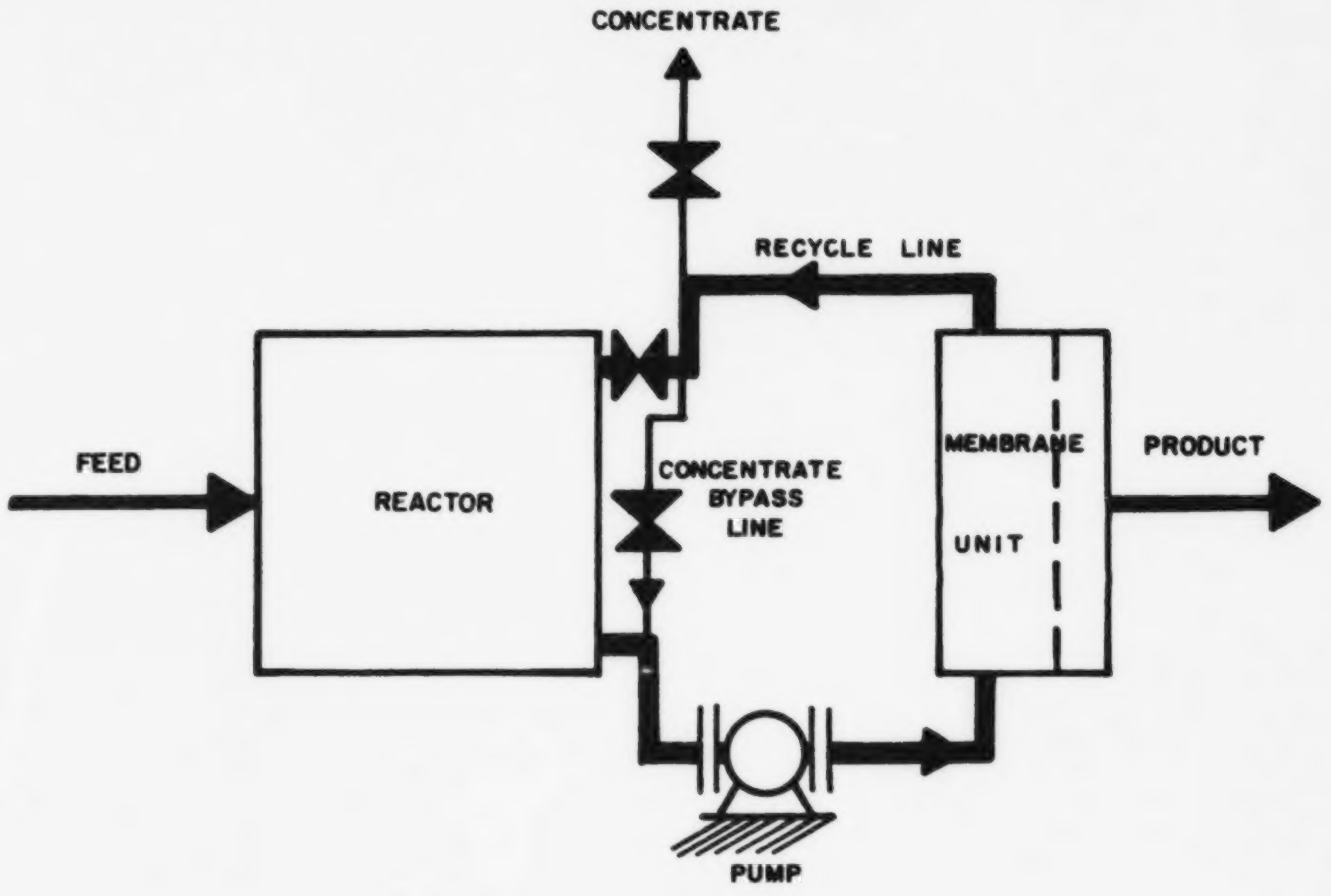
Membrane ultrafiltration is a pressure activated process using semi-permeable membranes which act as molecular screens to separate molecular and colloidal materials dissolved or suspended in a liquid phase.

Thus far, the principal commercial applications of the process have been in (1) industrial operations where valuable products can be recovered by separation from a bulk solution, (2) analytical application which provides a new method to separate molecules according to size and molecular weight, and (3) in a package waste treatment plant which separates mixed-liquor solids from a biological reactor. Solids are returned to the aerator and clarified product water is discharged.

A plant employing ultrafiltration has been installed on Pikes Peak to provide waste treatment and water reuse. The essentials of the process are shown in Fig. 13. The 15,000 gpd plant treats wastewater by high-solids activated sludge. The solids are separated in an ultrafiltration unit. Solids are returned to the aerator. Product water is excellent. Some typical removals are shown below.

Figure 12


BOD REMOVALS BY MBF SYSTEM
INFLUENT
EFFLUENT
BOD – mg/l
200
160
120
80
40
20
0
1800
2400
0600
1200
1800
2400
0600
SAMPLING TIME



ENZYME REACTOR

CONCENTRATE
RECYCLE LINE
FEED
REACTOR
CONCENTRATE BYPASS LINE
MEMBRANE
UNIT
PRODUCT
PUMP

FIG. 13 REACTION SYSTEM

Removals by Activated Sludge and Ultrafiltration

	Influent	Effluent	% Removal
BOD	382	<1	>99
COD	678	20	97
Turbidity	-	<0.1	-
SS	323	0	100
PO_4-P	12.2	7.7	37

These results emphasize that only molecules or particles greater than 500-1000 MW are separated. Inorganic ions such as phosphate are not retained. The 37% reduction of phosphate was due to biological uptake in cells which were removed.

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DEMINERALIZATION OF WASTEWATERS

Jesse M. Cohen

I. Introduction

A. Today's best conventional waste treatment processes were not designed to remove non-biologically degradable organics or inorganic salts. It appears certain that improved wastewater treatment will be based on the addition of physical-chemical separation processes to the current biological processes.

B. Degree of treatment required of a renovation process will be determined by the specific purpose for which the water is intended.

C. For certain industrial use and for deliberate municipal recycle, it will be essential to reduce the salt content of wastewater.

D. Need for demineralization is based on the fact that each water-use cycle results in an incremental addition of dissolved inorganic salts.

1. Average values for mineral increments through one municipal use are shown in Figure 1.

2. Increase of total dissolved solids ranges 100-500 mg/l.

3. Most important increments, in terms of magnitude, are sodium, sulfate, bicarbonate, and chloride.

E. Three principal methods are in varying stages of development designed to restore the mineral quality of sewage effluent to at least the quality of river water or better.

1. Electrodialysis

2. Reverse Osmosis

3. Ion Exchange

II. Electrodialysis

Electrodialysis is useful for partial demineralization of low mineralized waters, preferably not exceeding 2000 mg/l of TDS. Since municipal wastewaters generally do not exceed 1000 mg/l, this process is applicable.

A. Principles

1. Principle of operation of an electrodialysis unit is shown in Figure 2. When a direct electric voltage is impressed across a cell containing mineralized water, the positively charged ions (cations) migrate to the negative electrode and the negatively charged ions (anions) migrate to the positive electrode. If cation and anion-permeable membranes are placed alternately between the electrodes, alternate compartments become more concentrated in salts while the intervening compartments become more dilute.

2. Multiple compartments can be interconnected by manifolds which produce two products, one low in dissolved solids, one concentrated. Figure 3.

3. Membranes are composed of ion-exchange materials. The driving force for the migration of ions from solution to the membranes is electrical energy.

B. Operating problems

1. Stagnant films at the membrane surfaces lead to concentration gradients producing scaling from precipitation of compounds with low solubility.

2. Electrolysis of water, which produces hydrogen and hydroxyl ions, leading to increases in power consumption.

3. Deposition of colloidal negatively charged particles on the anion membrane causing fouling and decrease in demineralization.

C. Pilot plant operation

1. Experimental electrodialysis equipment was investigated, Figure 4, with the following conclusions:

    a. To maintain operation, the secondary effluent had to be pretreated for removal of organics by activated carbon and clarified by alum coagulation,

    b. pH of the concentrate stream was held at 5 or less to prevent calcium carbonate scale formation,

    c. Operation was very sensitive to fouling by colloids. Figure 5.

(1) Extended operation could be maintained only by reducing turbidity to 0.03 JTU. Higher turbidities produced short runs.

(2) Biological growth on the membranes contributed to fouling.

(3) Recovery from fouling could be obtained by shutting down for a day or so.

d. Blow-down amounted to about 10% of the product to prevent scaling.

2. Electrodialysis removes ions to varying degrees. Figure 6.

a. Selectivity for ions varies with time and with degree of fouling. Both phosphate and sulfate removal rates were affected.

3. Volume of waste concentrate will amount to ≈ 10% of product and methods for disposal of this concentrated waste must be found.

4. Firm costs for electrodialysis are not available. For a 10 mgd plant, capital cost to produce partially demineralized water from wastewater has been estimated at 0.34 dollars per gallon per day and operating costs at 16.1 cents per 1000 gallons. No provision is made in these costs for pretreatment and disposal of brine.

III. Reverse Osmosis

The theoretical capability of reverse osmosis to remove in excess of 90% of the inorganic ions, organic matter and colloids (bacteria and virus) makes this process an important discovery.

A. Principles

1. Figure 7. In normal osmosis when two liquids of differing salt concentrations are separated by a semi-permeable membrane, water will flow from the dilute solution to the more concentrated. Driving force is the concentration gradient.

2. In reverse osmosis pressure, in excess of the osmotic pressure, is applied to the concentrated side causing the normal flow of water to be reversed; i.e., from the concentrated to the dilute side of the membrane.

3. **Keys to the successful application of reverse osmosis to wastewater reclamation are:**

   a. Development of a membrane capable of reasonable service under operating conditions,

   b. Proper incorporation of the membranes into a system to maximize product flow at minimum cost.

B. Membranes

1. Practical reverse osmosis was achieved when Loeb and Sourirajan in 1960 developed a modified cellulose acetate membrane capable of water permeabilities some 500 times greater than earlier films.

2. Membranes consist of two distinct layers. One a spongy porous material which accounts for 99.8% of the thickness and an active layer which accounts for all of the separation of contaminants. Figure 8. Surface layer is typically 0.25 μ thick, porous layer 100μ thick.

3. Original theory of operation considered a "straining" mechanism. Current theory is that water molecules dissolve in the membrane material and then diffuse through it. Inorganic ions have lesser solubility in the membrane and have restricted mobility.

C. Water throughput-flux

1. Flux is defined as the flow of water through a membrane, measured in gallons per square foot per day.

2. Flux is directly proportional to applied pressure - minus the osmotic pressure - and inversely proportional to thickness of membrane.

3. Relationship of pressure and salt rejection (solute retention) are shown in Figure 9.

   a. At low pressures, low flows and little salt removals are obtained,

   b. As pressure exceeds osmotic pressure, flow and salt rejection increase and 100% salt rejection is approached asymptotically.

4. Fluxes in the range of 20-40 gfd are required to make RO economically attractive. Current fluxes of ≈ 10 gfd at 750 psi are common.

5. A major current problem is the inability to maintain flux because of fouling and chemical and physical changes that occur in the membrane.

D. Materials rejected

1. Inorganic salts are rejected in amounts exceeding 90%.

2. Organic molecules, with the exception of certain low molecular weight materials like amines, alcohols and acids, are removed.

3. Suspended and colloidal materials including bacteria, virus and sewage colloids are completely removed.

E. Membrane separator design

A key factor in practical operation of RO is the mechanical design of the unit.

1. Plate and frame, using flat membrane sheets in a device similar to a filter press was one of the earliest designs. Figure 10.

a. Units capable of producing 100,000 gal/day have been developed by Aerojet-General. Largely used for brackish water treatment.

2. A simple tubular design consists of porous tubes which are lined with cellulose acetate membranes. Design is similar to heat exchangers. Flow is from inside the tubes at pressure and discharged from the outside surfaces of the tubes at ambient pressure. Figure 11.

3. To obtain a maximum of membrane area in a small volume, a "spiral-wound" design was developed by General Atomics. Several hundred feet of membrane can be accommodated in a cubic foot of pressure vessel volume. Figure 12.

4. The ultimate in greatest area per unit volume is approached by "hollow-fiber" design.

a. About 20 million hollow fibers can be packed into a shell 1 foot in diameter and 7 feet long.

b. A 12-inch permeater 7 feet long contained in a unit only 5.5 cu.ft. in volume provides about 50,000 square feet of membrane surface and will produce 7500 gal/day.

c. Flux with these fibers is small - currently about 0.15 gal/ft^2/day. Fluxes of 1.5 gfd may be attainable but this will still be 1/10 that obtainable with other designs.

d. This configuration can be characterized as "very high surface area, low flux permeater."

F. Cost

Application of RO to wastewater is still in early stage of development.

1. Based on experience in treating brackish water, cost estimates have ranged from 30-60 cents/1000 gallons.

2. These problems remain to be solved before RO can be successfully applied to wastewater.

a. Increased fluxes.

b. Maintenance of flux in the face of fouling and physical and chemical changes occurring at the membrane.

c. Increased product to waste ratios. Up to 90% has been obtained.

d. Disposal of concentrate.

IV. Ion Exchange

Ion exchange has been used for many years in water treatment for softening and boiler water conditioning. Development of newer types of resins which are not fouled by organics and newer techniques of using these resins demonstrate that ion exchange is a practical method for demineralization of wastewater.

A. Principle

1. Ion exchange materials, both natural and synthetic, are absorbents which carry charged ionic groups. To maintain electroneutrality, each ionic site must have associated with it an ion of opposite charge (counter-ion). The success of ion exchange depends on the ability of the counter-ion to be replaced or exchanged for another ion of the same charge.

2. When all counter-ions have been replaced, the ion exchange material is exhausted. Regeneration is obtained by contacting the exchanger with a concentrated solution of the original counter-ion.

B. Classification

1. Ion exchangers are classified by the charge of the exchangeable ion. Thus acid or cationic resins will exchange cations like Ca^{++} and Mg^{++} and base or anionic resins will exchange OH^- or Cl^- anions.

2. Classification also considers the degree of dissociability of the active group. Thus:

a. Strong acid resins (cationic) contain sulfonic acid groups.

b. Weak acid resins (cationic) contain carboxyl or phenolic groups.

c. Strong base resins (anionic) carry quaternary ammonium groups.

d. Weak base resins (anionic) usually have amine groups which are only slightly ionized in the OH form.

3. A common matrix material of synthetic resins is styrene or vinyl benzene which has been polymerized and which have ranging functional groups attached. Figure 13.

a. A very wide variety of resins can be prepared to meet specific needs by varying functional groups, degree of ionization, and physical characteristics.

C. Selectivity

1. A general purpose resin will exchange any of the common ions but, in a mixture of ions, selectivity will occur.

a. For strong cation exchangers the order of selectivity is $Ca^{++} < K^+ < NH_4^+ < Na^+$

b. For weak anion exchangers the order of selectivity is $SO_4^= < HPO_4^= < NO_3^- < Cl^-$

D. Wastewater demineralization

For demineralization, both an anion and cation exchanger is required, consisting of a weak anion and a strong or weak cation exchanger.

1. For economic demineralization, these requirements for the process must be met.

a. Capability of regeneration at theoretical efficiencies using low cost acids and bases.

b. High operating exchange capacities.

c. Ease of removal of regeneration chemicals with a minimum of rinse water.

d. Long life of resin to physical and chemical attrition.

e. Waste regenerants should pose a minimum disposal problem.

E. Desal process developed by Rohm and Haas has promise for economic demineralization of wastewater.

1. One modification consists of a 3-bed design consisting of a weak anion, a weak cation and a weak anion exchanger in series. Figure 14.

a. First column converts all anions to the bicarbonate form. Removes all other anions.

b. Second column converts bicarbonates to CO_2 and removes cations.

c. Third column in OH^- form absorbs CO_2 to become the bicarbonate form. This column then becomes the first in the series after columns 1 and 2 have been regenerated with ammonium hydroxide and sulfuric acid respectively.

2. A modification more applicable to wastewater treatment is shown in Figure 15.

a. Weak anion exchanger, in the bicarbonate form, removes all anions: SO_4^{2}, $PO_4^{=}$, NO_3 and Cl^-.

b. Column effluent is polymer flocculated, CO_2 removed and lime softened which removes: suspended solids, Ca^{++}, Mg^{++}, and Fe^{+++}.

c. Final column, a weak cation exchanger, removes remaining cations: NH_4^+, Na^+ and K^+.

d. Exchangers regenerated with ammonium hydroxide and sulfuric acid. Calcining of lime would provide CO_2 for conversion of anion exchanger to bicarbonate form.

e. About one-half of the COD is removed by the anion exchanger and quantitatively removed by the regenerant.

f. Final product, containing residual amounts of inorganic solids - ≈ 50 ppm - and 1/2 of the original COD, could be blended with undemineralized water to reduce cost.

F. Cost

Process has not been applied on a sufficiently large scale to obtain reliable cost data.

1. Estimate for 1 and 10 mgd plants, including capitalization, equipment and chemical costs for the modified Desal process is 17-18 cents/1000 gallon.

   a. CO_2 is assumed to be available.

   b. Resin life estimated to 3 years.

   c. No costs included for ammonium recovery and brine disposal.

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FIG. 1

AVERAGE COMPONENTS IN DOMESTIC SEWAGE EFFLUENTS

CONSTITUENT	APPROX BUILD-UP THROUGH ONE MUN USE	NORMAL RANGE OF BUILD-UP	REPRESENTATIVE SECONDARY EFFLUENT
Ca ($CaCO_3$)	30 ppm	15- 40	95
Mg ($CaCO_3$)	13	20-40	39
Na	55	40-700	84
SO_4	25	10- 40	51
CL	35	20-125	50
PO_4	30	15-40	30
NO_3	8	0- 18	8
SiO_2	15	10- 20	35
ABS	3	1- 4	3
COD	70	40- 40	70
BOD	15	9- 40	15
TDS	250	100-500	550
HCO_3	65	---	170
pH	---	---	7.5

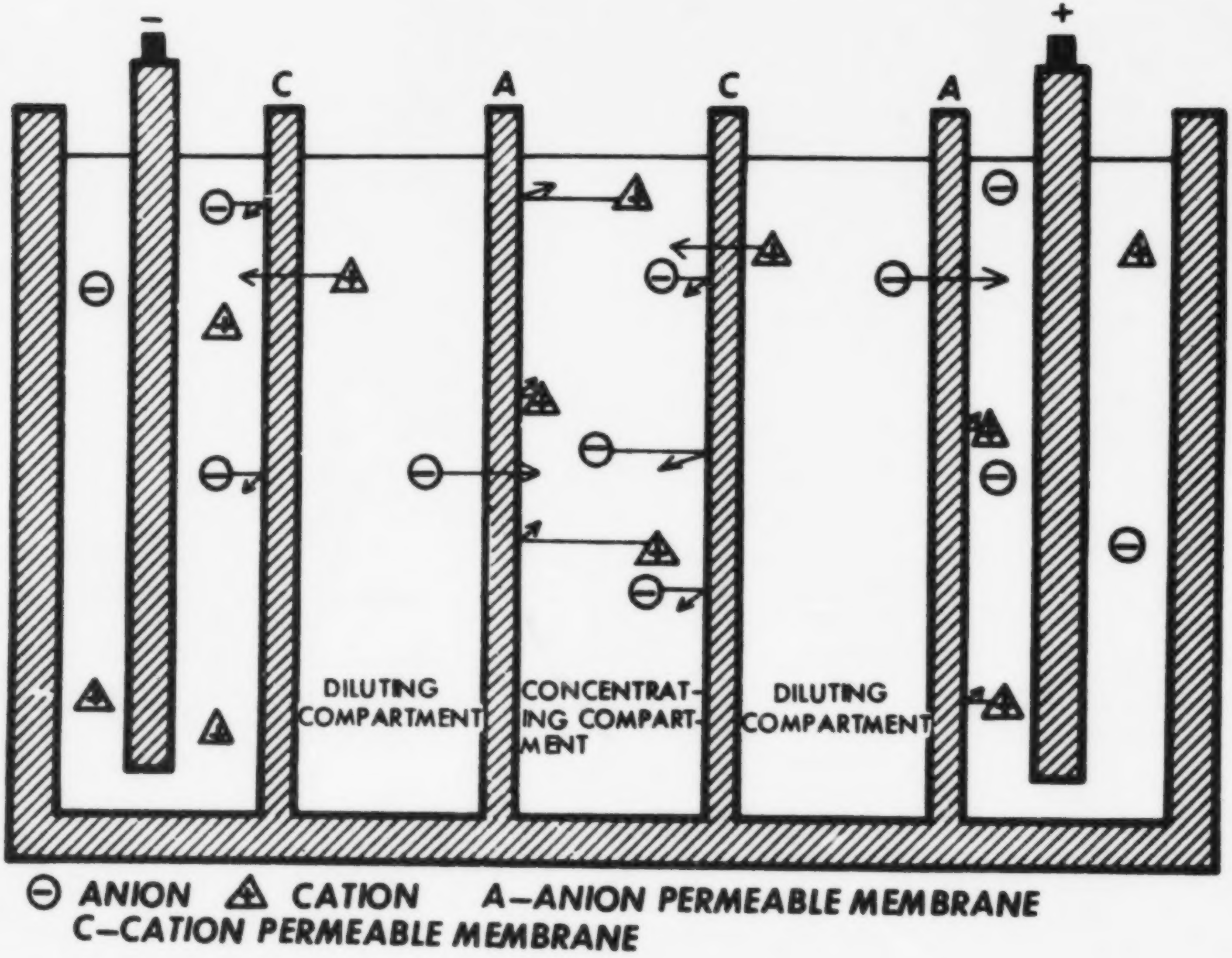
-
+
C
A
C
A
DILUTING COMPARTMENT
CONCENTRATING COMPARTMENT
DILUTING COMPARTMENT
⊖ ANION
CATION
A—ANION PERMEABLE MEMBRANE
C—CATION PERMEABLE MEMBRANE

FIGURE 2 &
FIGURE 3. ELECTRODIALYSIS PRINCIPLE

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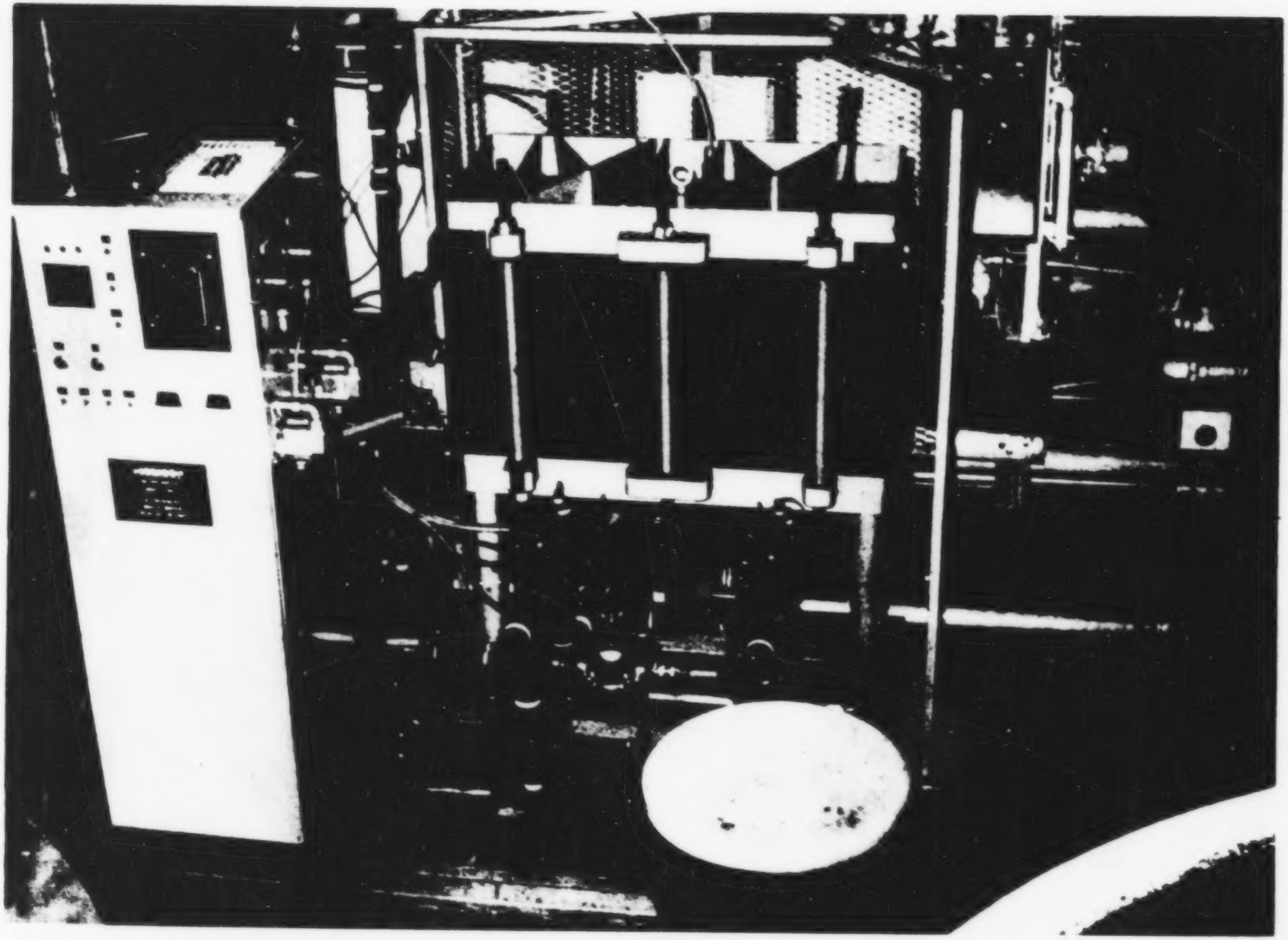


FIGURE 4

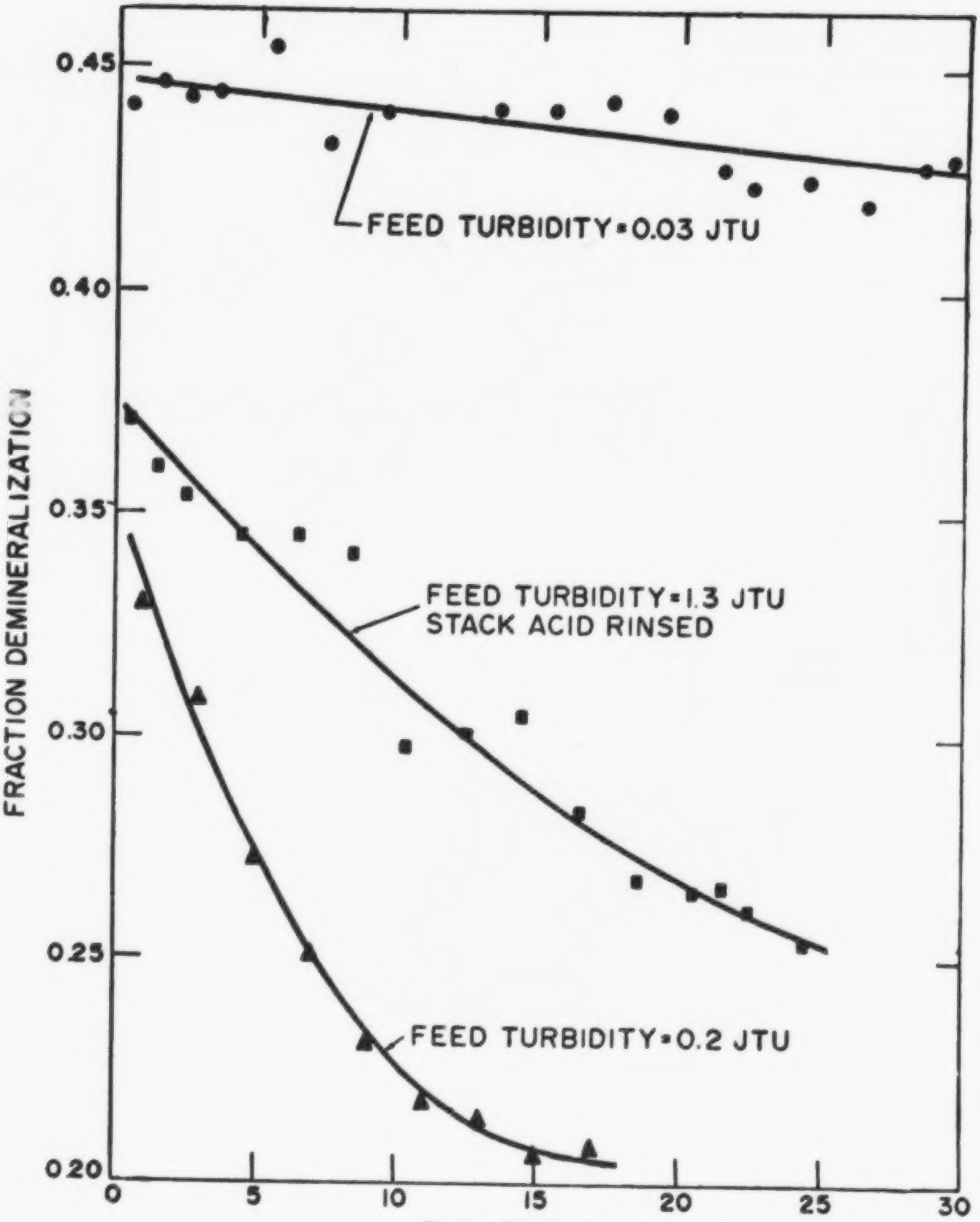


FIG. 5 **Effect of turbidity on demineralization.**

FIG. 6

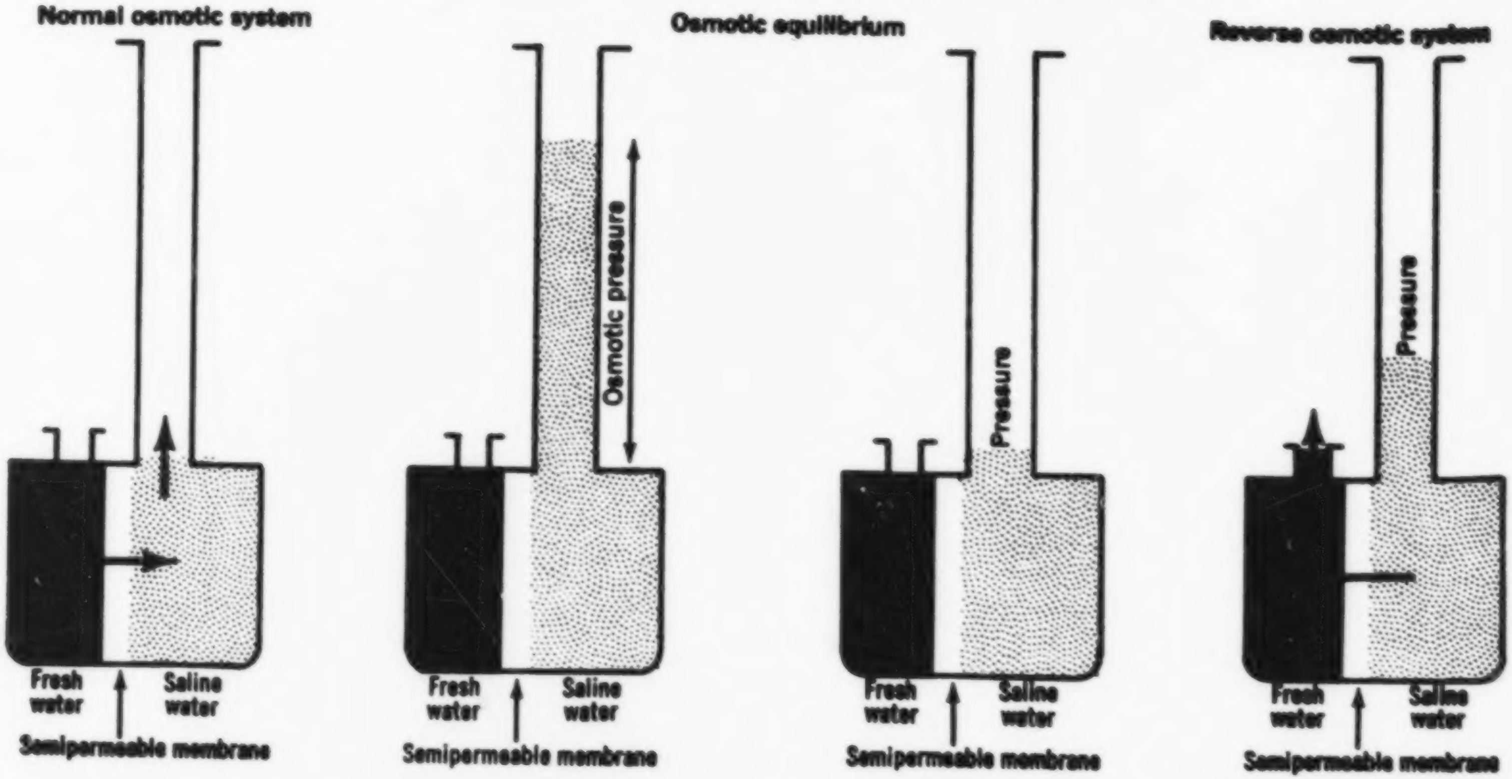
Ion Removal Selectivity

Ion	Fraction of Ion Removed / Fraction of All Ions Removed	
	Average	Range
Bicarbonate	0.77	0.63–0.95
Chloride	1.25	1.07–1.51
Nitrate	1.22	1.02–1.48
Phosphate	0.72	0.25–1.03
Sulfate	1.11	0.21–2.00
Ammonium	1.15	1.05–1.3
Calcium	1.25	1.01–1.48
Magnesium	0.98	0.71–1.35
Potassium	1.14	0.77–1.35
Sodium	0.79	0.52–0.99
Calcium and magnesium	1.13	1.05–1.28

GPO 821-571-8

FIG. 7

Pressure on osmotic system leads to reverse osmosis


Normal osmotic system
Fresh water
Saline water
Semipermeable membrane
Osmotic pressure
Fresh water
Saline water
Semipermeable membrane
Osmotic equilibrium
Pressure
Fresh water
Saline water
Semipermeable membrane
Reverse osmotic system
Pressure
Fresh water
Saline water
Semipermeable membrane

Reproduced from best available copy.

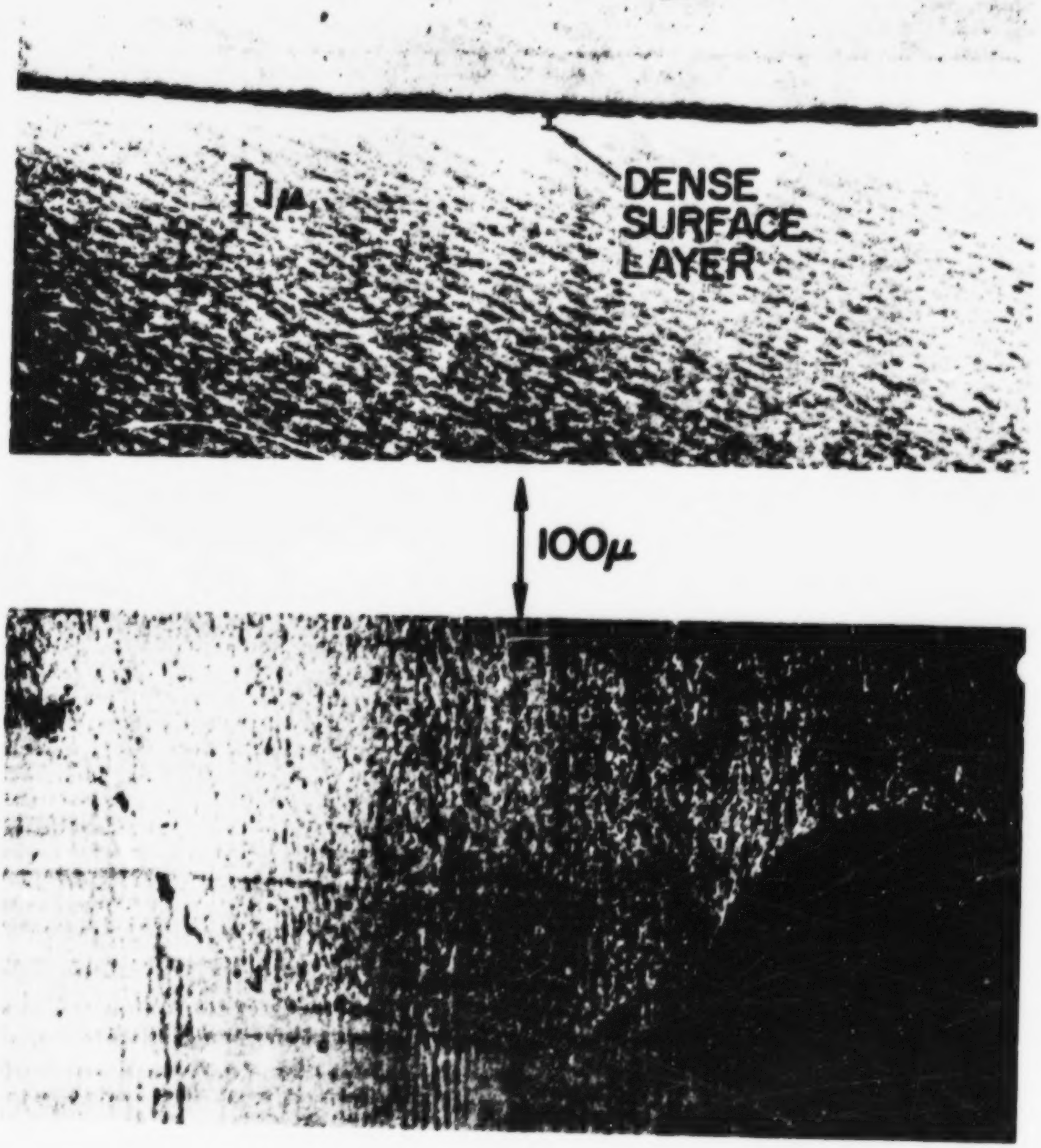

DENSE SURFACE LAYER
1μ
100μ

FIG. 8 COMPOSITE OF TWO ELECTRON PHOTO-MICROGRAPHS OF CROSS SECTIONS OF MODIFIED CELLULOSE ACETATE MEMBRANE

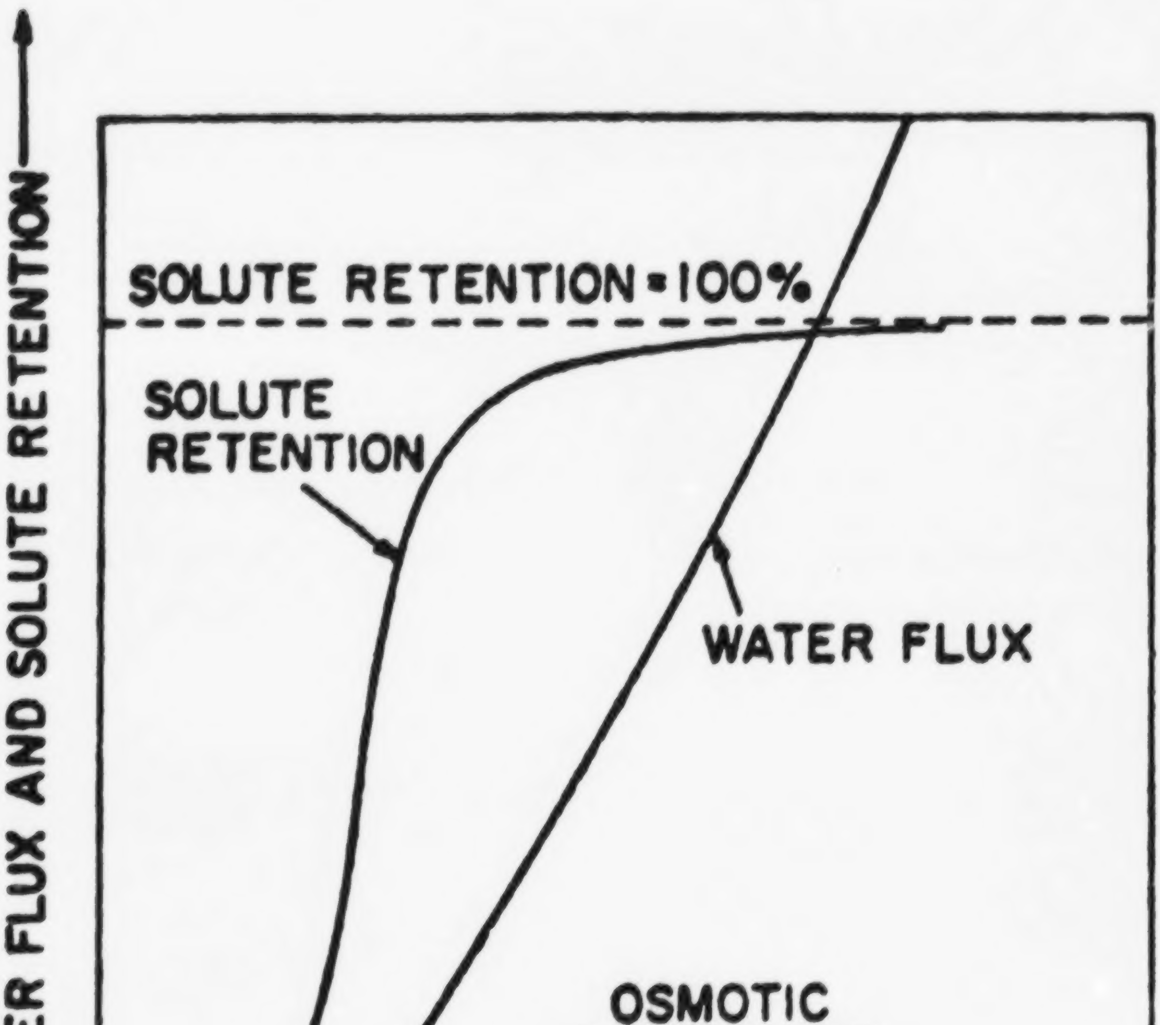


FIG. 9 **SYSTEM PRESSURE →**

Solute retention and water flux in idealized membrane.

MULTIPLE PLATE TYPE DESALINATION CELL

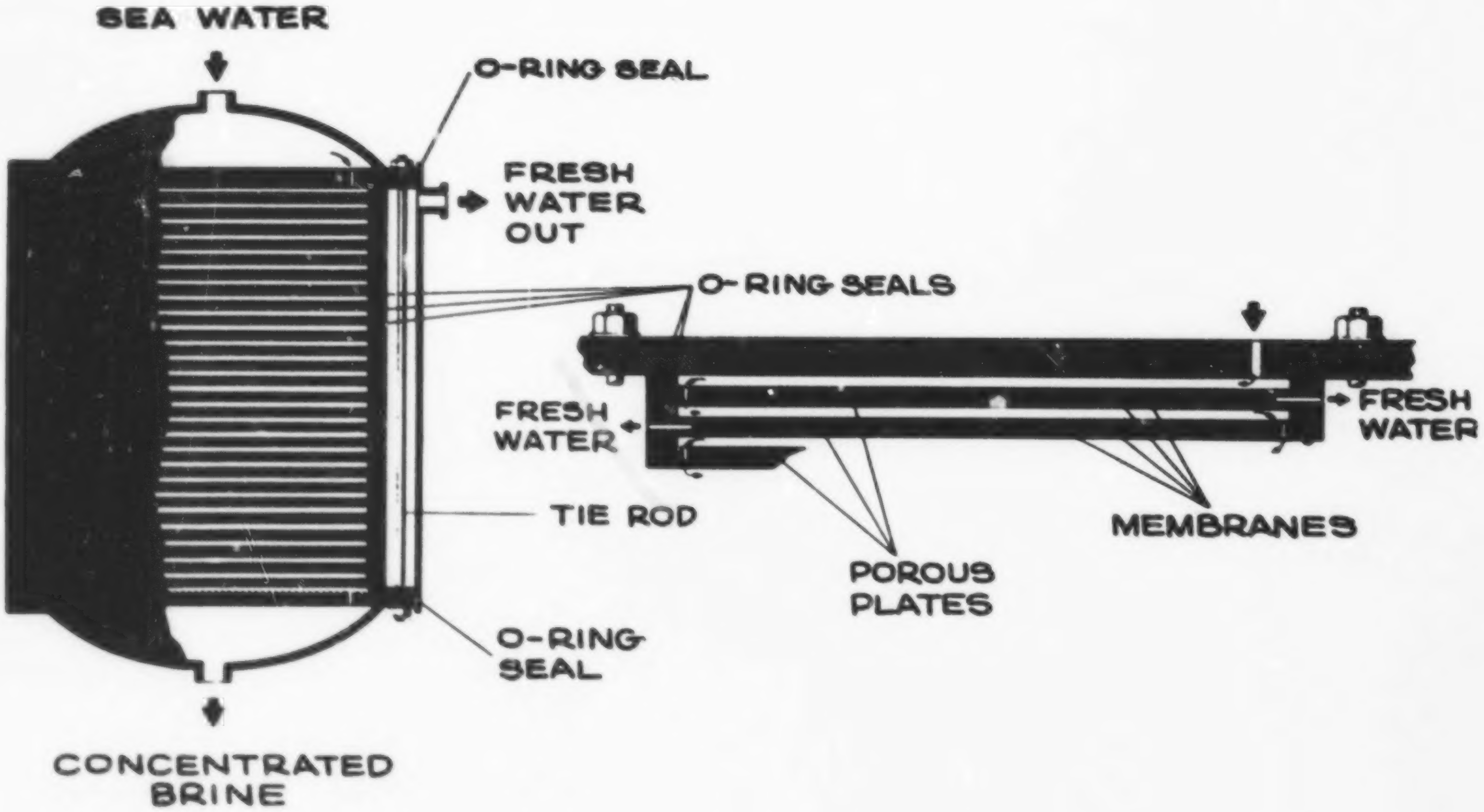
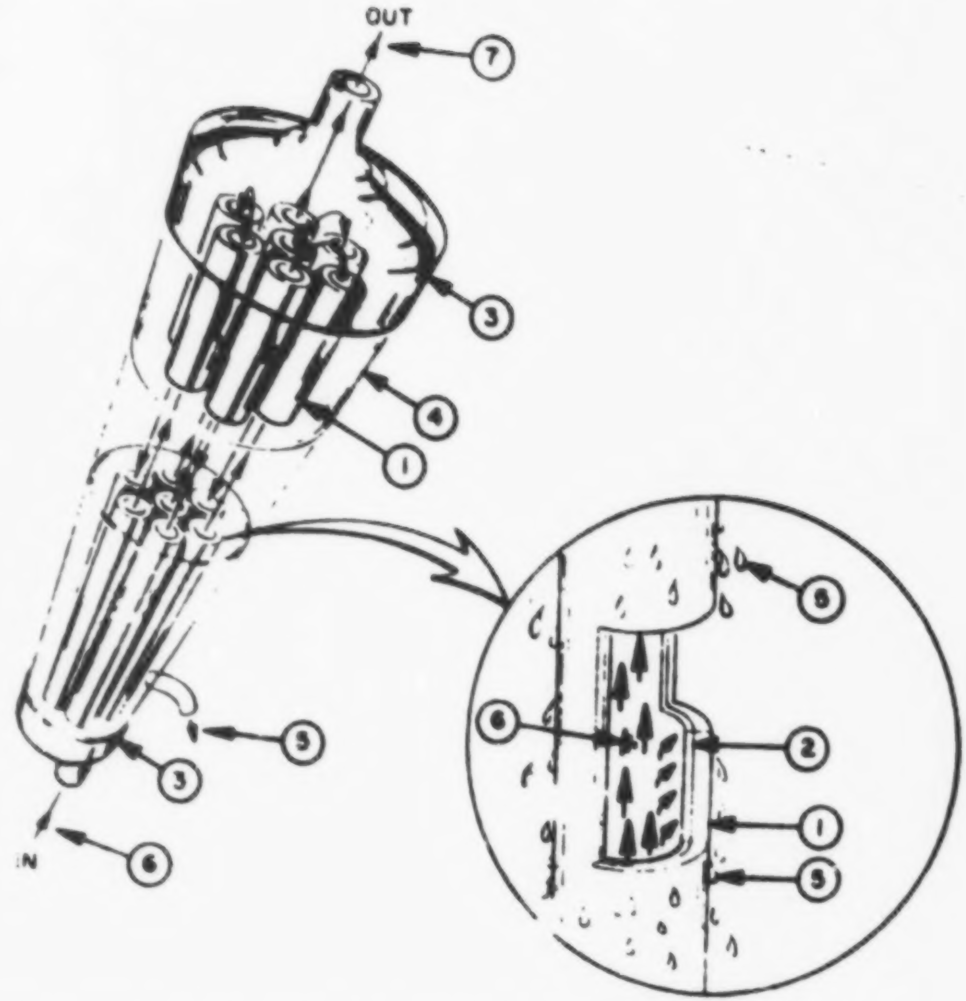

SEA WATER
O-RING SEAL
FRESH WATER OUT
O-RING SEALS
FRESH WATER
FRESH WATER
TIE ROD
MEMBRANES
POROUS PLATES
O-RING SEAL
CONCENTRATED BRINE

FIGURE 10


OUT
7
3
4
1
5
3
6
IN
8
6
2
1
5

(1) FIBERGLASS TUBE
(2) OSMOTIC MEMBRANE
(3) END FITTING
(4) PVC SHROUD
to collect product water
(5) PRODUCT WATER
(6) FEED SOLUTION
(7) EFFLUENT

Figure 11. A Tubular Reverse Osmosis Unit

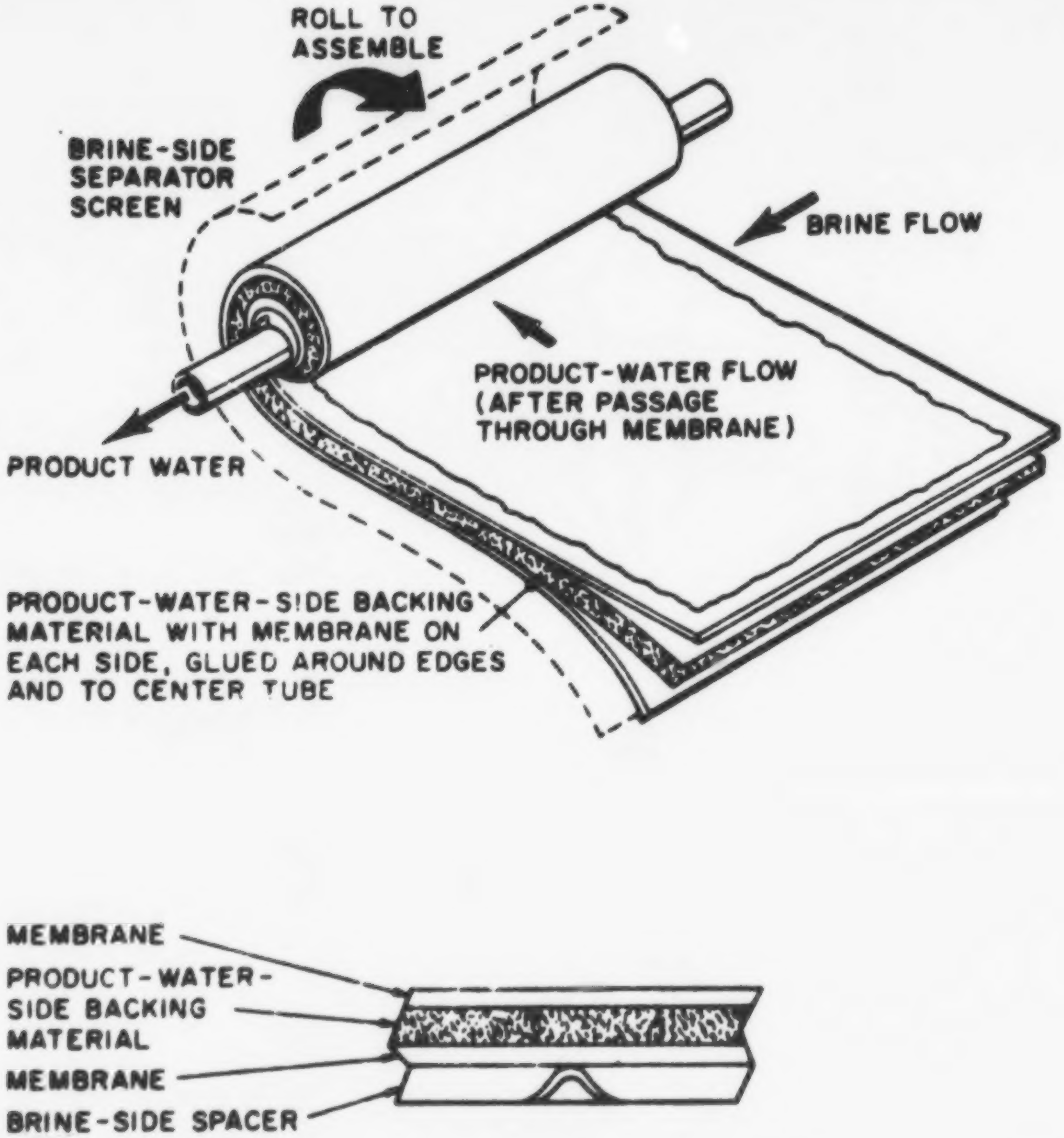
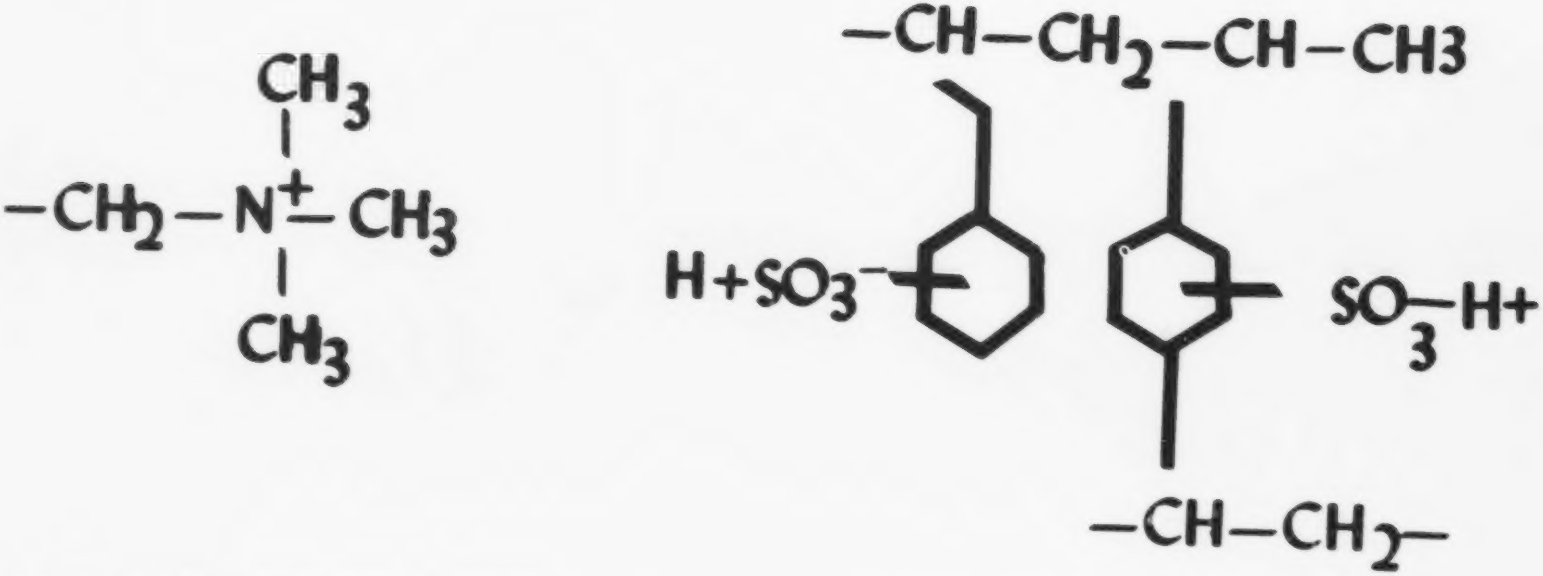

ROLL TO ASSEMBLE
BRINE-SIDE SEPARATOR SCREEN
BRINE FLOW
PRODUCT-WATER FLOW (AFTER PASSAGE THROUGH MEMBRANE)
PRODUCT WATER
PRODUCT-WATER-SIDE BACKING MATERIAL WITH MEMBRANE ON EACH SIDE, GLUED AROUND EDGES AND TO CENTER TUBE
MEMBRANE
PRODUCT-WATER-SIDE BACKING MATERIAL
MEMBRANE
BRINE-SIDE SPACER

Figure 12. A Spiral-Wound Reverse Osmosis Module

Reprinted with permission from General Dynamics, General Atomic Division



$-CH_2-N^{\pm}-CH_3$
CH_3
CH_3
$-CH-CH_2-CH-CH3$
$H+SO_3^-$
SO_3-H+
$-CH-CH_2-$

STRONG BASE RESIN

STRONG ACID RESIN

FIG. 13

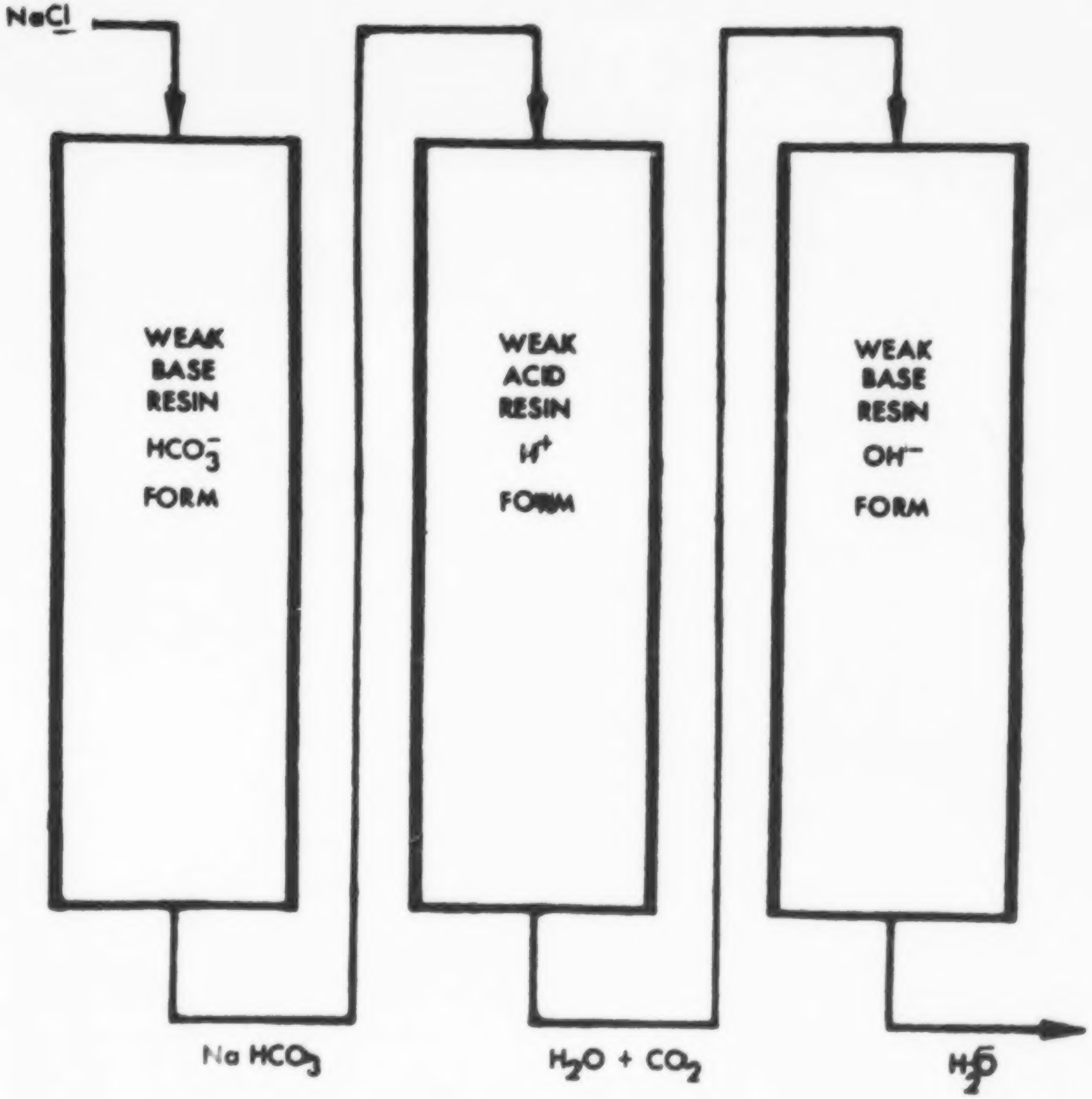

NaCl
WEAK BASE RESIN
HCO₃⁻ FORM
WEAK ACID RESIN
H⁺ FORM
WEAK BASE RESIN
OH⁻ FORM
Na HCO₃
H₂O + CO₂
H₂O

Figure 14. DESAL Process

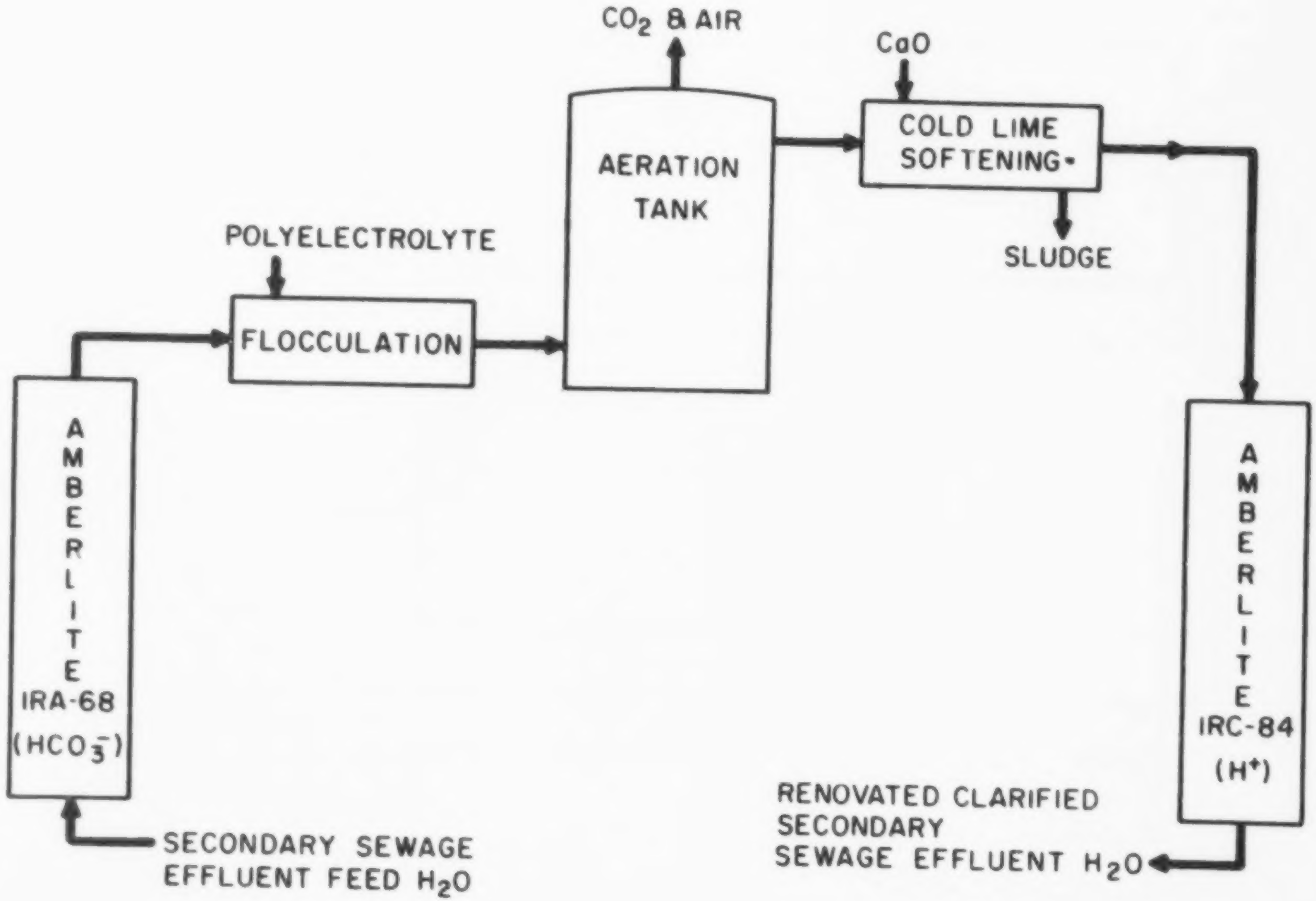

CO2 & AIR
CaO
COLD LIME SOFTENING
AERATION TANK
POLYELECTROLYTE
SLUDGE
FLOCCULATION
AMBERLITE IRA-68 (HCO3-)
AMBERLITE IRC-84 (H+)
RENOVATED CLARIFIED SECONDARY SEWAGE EFFLUENT H2O
SECONDARY SEWAGE EFFLUENT FEED H2O

Figure 15. Flow diagram of ion exchange process for renovation of secondary sewage effluent

FIG. 16

Cost Estimate for Renovation of Haddonfield, N. J., Secondary Sewage Effluent

	$ Cost/1000 Gal.	
	1,000,000 gal./day	10,000,000 gal./day
Alkalization	0.0655	0.0631
Dealkalization	0.0500	0.0478
Carbonation	0.0030	0.0030
Degasification	0.0042	0.0029
Flocculation (chemicals only)	0.0436	0.0436
Lime softening	0.0158	0.0158
Grand total	$0.1821	$0.1762

CURRENT STATUS OF ADVANCED WASTE-TREATMENT PROCESSES

JULY 1, 1970

ADVANCED WASTE-TREATMENT RESEARCH LABORATORY

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

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FOREWORD

Waste treatment technology is moving rapidly nowadays. A huge impetus has been given to this field by the substantial sums of money made available for research by the Congress and administered by the Research and Development Office of the Federal Water Quality Administration. The Advanced Waste Treatment Research Laboratory (AWTRL) in Cincinnati, Ohio is a key element in conducting treatment research for FWQA.

This status report is current as of July 1, 1970. It reports the programs of the Advanced Waste Treatment Research Laboratory but mentions other pertinent work as well. It is not, however, a comprehensive review of the field. The purpose of the report is to inform Federal Water Quality Administration operating and managing officials of the state of the art of treatment. It is expected others will find it useful. If the details of the scientific investigations are desired, they are available in various reports named in the text.

F. M. Middleton
Director of Research
Advanced Waste Treatment
Research Laboratory

ADVANCED WASTE TREATMENT RESEARCH LABORATORY

Cincinnati, Ohio

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

PPB 1101 & 1105	Municipal Pollution Control
PPB 1700	Waste Treatment and Ultimate Disposal Technology
PPB 1603	(Biological Identification of Pollutants) Virus Studies

DIVISION OF PROCESS RESEARCH & DEVELOPMENT
FEDERAL WATER QUALITY ADMINISTRATION
U. S. DEPARTMENT OF THE INTERIOR

GPO 931-871-10

ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1. 1970

MUNICIPAL POLLUTION CONTROL TECHNOLOGY
SEWERED WASTES

PPB 1101

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

MUNICIPAL POLLUTION CONTROL TECHNOLOGY
PPB - 1101 - SEWERED WASTES

APPLICATION OF ADVANCED WASTE TREATMENT PROCESSES TO THE TREATMENT OF MUNICIPAL WASTEWATER

Transfer of advanced waste treatment technology from laboratory scale or experimental pilot plant scale is taking place through the continuing evaluation and development of specific processes and treatment systems in large-scale pilot plants and full-scale demonstration plants. The following discussion covers specialized treatment processes that have achieved full-scale application. Comments are made as to the general effectiveness and limitations or disadvantages of such processes under actual use conditions.

PURE OXYGEN IN ACTIVATED SLUDGE PROCESS

Promising results were obtained from a study recently completed at Batavia, New York in which the use of pure oxygen was compared with air in the activated sludge process. The test plant has two identical and separate 1.25 mgd trains. Each treatment train has separate aeration tanks, final sedimentation tanks, and return sludge facilities. Primary sedimentation is not provided. One train was covered and converted to use of pure oxygen and operated in parallel with the air system. With recent developments in oxygen production and dissolution technology, under the conditions of the Batavia test, pure oxygen was shown to be competitive with air. The oxygen can be produced economically on site.

Under test conditions the pure oxygen train achieved 90 percent or more BOD removal at detention times of 1 to 1.5 hours. With 3 hours detention time BOD removal averaged 85 percent for the air train and 93 percent for the oxygen train. Another significant difference in performance was quantity of waste activated sludge. Although confirming data are needed, preliminary results indicate a reduction of 30-40 percent. Further information will be obtained during a continuation of the study.

Based on the Batavia data, cost estimates projected for new plants indicate the possibility of lower capital investment and operating costs for the pure oxygen treatment. The major factor contributing to the cost reduction is the ability to carry higher MLSS and thereby reduce aeration tank capacity to 40 or 50 percent of that required for the conventional systems. Additional savings are indicated for sludge handling and disposal.

Results of the study are being published as an FWQA research report titled "An Investigation of the Use of High Purity Oxygen Aeration in the Conventional Activated Sludge Process." Copies of the report, expected to be ready by September 15, 1970, may be obtained by writing Planning and Resources Office, Office of Research and Development, FWQA, Department of Interior, Washington, D. C. 20242.

Further evaluation and development of the pure oxygen process will be accomplished under terms of an FWQA R&D Grant recently awarded to New York City. A 20 mgd train at the Newtown Creek treatment plant will be converted to the use of pure oxygen and operated for at least 12 months.

The use of pure oxygen for the treatment of municipal waste waters is being aggressively promoted by Linde Division of Union Carbide Corporation who was contractor for FWQA on the Batavia study.

GRANULAR ACTIVATED CARBON

The use of granular activated carbon for removal of nonbiodegradable organics, color and residual BOD has been demonstrated in full-scale plants and is felt to be sufficiently developed to be used on full-scale applications wherever conditions warrant such treatment. Because suspended solids are partially removed on the carbon, the solids load and need for pretreatment must be considered when designing a carbon adsorption system. In order for the carbon treatment system to be economical, the used carbon must be regenerated and reused. Large-scale plants currently using and regenerating granular activated carbon include the 7.5 mgd plant at Lake Tahoe and the 0.5 mgd plant at Nassau County, New York. A number of articles have been published covering the Tahoe plant. One appeared in the June, 1969 issue of Civil Engineering entitled "Wastewater Reclamation and Export at South Tahoe." A few copies are available from the Cincinnati Laboratory.

At the 300,000 gpd Pomona, California Pilot Plant secondary effluent is applied directly to the carbon columns without requiring excessive backwashing. This is only possible, however, because of exceptionally high quality secondary effluent at this location. Results of this study will appear in a 1970 issue of the Chemical Engineering Progress Symposium Series.

Full-scale evaluation of granular carbon adsorption as a replacement for biological treatment will be obtained on a 10 mgd plant at Rocky River, Ohio. This is an R&D Grant project which involves chemical pretreatment of raw sewage in an improved primary treatment ahead of the carbon columns. Construction of this plant is scheduled for completion by fall of 1971. Further details regarding design and operating conditions scheduled for this plant may be obtained from Mr. A. N. Masse, Cincinnati, Ohio.

PHOSPHORUS REMOVAL

Phosphorus removal from wastewater on plant scale has been carried out for a number of years at certain locations where the water was needed for industrial reuse purposes. It has been relatively recent, however, that phosphorus removal has been considered necessary as a pollution control measure.

Although purely biological methods of phosphorus removal have been proposed, it appears that addition of chemicals to the water to precipitate the phosphorus is the only dependable method. Chemicals that can be used are iron salts, aluminum salts, and lime. The simplest method for carrying out chemical precipitation is to add the chemicals at some point in a conventional activated sludge plant. The point of addition can range from before primary treatment to near the exit of the aerators. Iron salts and aluminum salts are preferable to lime. A number of R&D Grant projects have been sponsored by FWQA including Grand Rapids, Michigan at 45 mgd. Contact Mr. E. F. Barth, Cincinnati, Ohio for additional information.

An alternative method for carrying out precipitation of phosphorus is by using a clarifier-settler combination either for treating screened raw sewage or as a tertiary treatment. Lime presently appears most appropriate for this method of removal. Excellent phosphorus removal can be obtained along with a high degree of solids removal, especially if the settler is followed by a filter. Probably the best known example of tertiary chemical clarification is the 7.5 mgd plant at Lake Tahoe. Reference has been made to this plant in connection with carbon treatment. Other FWQA supported projects include plants at Colorado Springs and Nassau County. When chemical clarification is used for treatment of raw sewage, it can serve as the first stage of a purely physical-chemical treatment system. Under an R&D Grant, a 5 mgd plant will be constructed and operated at Painesville, Ohio that utilizes chemical clarification followed by carbon treatment. There is increasing interest in this type of treatment system. Additional plants are likely to be constructed in the near future.

AMMONIA STRIPPING

Up to 95 percent of the ammonia in wastewater can be air-stripped from solution using about 400 ft^3 of air per gallon of water treated. Effective ammonia removal requires a pH of 11. A nitrified secondary effluent cannot be treated by this method.

A 3½ mgd ammonia stripping tower has been operated at South Lake Tahoe to treat one-half of the total flow at that location. This is part of a two-stage lime precipitation process. Lime is added in the first stage to raise the pH to 11. This step removes most of the suspended solids, phosphates, and carbonate compounds. Effluent from the first-stage clarifier is then subjected to countercurrent air contacting to remove ammonia; effluent from the stripping tower is recarbonated with CO_2 to precipitate excess calcium as $CaCO_3$ in the second-stage clarifier; at this point the pH is dropped to 9.5.

Although the process has been quite effective in reducing the ammonia content of the wastewater, operational problems raise questions as to the desirability of promoting the widespread use of ammonia stripping towers. The process is subject to freezing problems in cold climates and reduction of the ammonia removal efficiency at low temperatures. Lime deposits on the slats and superstructure of the tower create serious maintenance problems. At this time, use of stripping towers will most likely be restricted to more temperate locations where freezing is not a problem or areas where high percentage removal is not required during winter months.

POLYMER ADDITION TO PRIMARY SEDIMENTATION

The addition of polymers to raw sewage to improve sedimentation of suspended solids was studied at the District of Columbia Water Pollution Control Plant. The study was carried out at full scale on this 240 mgd plant. The test involved three separate phases in which each of three polymer suppliers carried out extended studies with his own most effective polymer. Best results were obtained with anionic polymers used in doses of less than 1 mg/l. The amount of solids removed by sedimentation increased by as much as 25 percent. Because of hydraulic overload at the plant, which affected operation of the final settlers, the improved primary treatment did not improve overall treatment significantly. For plants having only primary treatment, however, use of polymers could increase effectiveness of treatment significantly. Use of polymers to improve primary treatment may also improve secondary treatment where organic load to the aerators is very high. Additional results of the study will be reported in an FWQA research report entitled "Raw Wastewater Flocculation with Polymers at the District of Columbia Water Pollution Control Plant." Copies are expected to be available from FWQA's Research Division by October 1, 1970.

COMBINATION CONVENTIONAL-AWT TREATMENT

The treatment plant at South Tahoe, California is probably the best known advanced treatment plant in the country. Reference has already been made to the plant several times. It is of 7.5 mgd capacity and includes conventional primary treatment and activated sludge treatment followed by tertiary processes including two-stage lime clarification with ammonia stripping between stages, pressure multimedia filtration and granular carbon treatment. The effluent from the plant is of high clarity and contains only traces of phosphorus and organic materials. The water is presently exported to Nevada for eventual use in irrigation of crops after prior holding in a recreational lake, Indian Creek Reservoir. The lake can be used for all types of water recreation including contact sports. Use has been restricted, however, by a lack of facilities. Because of the high quality of the water, algae are not a significant problem.

Since biological treatment is included in the Tahoe system, the system is a hybrid between conventional and purely physical-chemical advanced treatment. There is increasing interest in pure physical-chemical systems. These have the advantage of not being affected by toxic materials that can upset the operation of biological processes for long periods. They also require less land area. Where only organic and phosphorus removal is required, chemical clarification followed by carbon treatment are the processes considered most reliable at this time. R&D Grants at Rocky River and Painesville, Ohio utilize variations of these processes. The effluents from these plants will not be of as high a quality as that from the Tahoe plant. The effluents are expected to be the equivalent of secondary effluent. Where nitrogen removal is required, ammonia stripping or selective ion exchange are most likely candidates. Stripping is less expensive but is subject to several operating problems. More work is needed to develop a completely satisfactory physical-chemical nitrogen removal system.

DESIGN MANUALS FOR ADVANCED WASTE TREATMENT PROCESSES

We have recognized the need for improved methods of disseminating results from research and development programs and we are currently making arrangements for a series of treatment process design manuals. These manuals will supplement technical seminars and publications covering results of the Advanced Waste Treatment Laboratory inhouse and extramural projects.

The information needed by consulting engineers for design of advanced waste treatment plants is now available for some processes.

However, the data are contained in numerous segmented publications, reports and manufacturers' literature. The purpose of the manuals will be to compile available information in a form which can be readily utilized, and provide detailed information on hardware selection and system design.

Contracts are being negotiated for preparation of the following design manuals:

1. Activated carbon adsorption, primarily for the treatment of secondary effluent or an equivalent waste stream.

2. Phosphorus removal by chemical treatment and solids separation.

3. Suspended solids removal including such techniques as microscreening, filtration, tube settlers, etc.

4. Upgrading of existing plants through the application of such techniques as pure oxygen treatment, flow equalization, etc.

Target date for completing the first manuals is March, 1971. At that time the manuals should be ready for general distribution to consulting engineers and other treatment plant design interests.

PPB - 1101 - SEWERED WASTES

Flow Reduction From Individual Homes and Alternative Waste Collection Systems

FLOW REDUCTION

A study on state-of-the-art of methods for flow reduction has been completed by Electric Boat Division of General Dynamics Corporation. This study concluded that use of modified plumbing fixtures for flow reduction is economically feasible and would result in cost savings in many cases. There are many household functions in which water is used wastefully. Water usage could be reduced up to 35 percent by use of presently available devices and technology. Feasible devices are shallow trap toilets, toilets with separate flush cycles for urine and feces, flow control showers, and faucet aerators. Treatment and reuse of wastewater in individual homes is not economically feasible, except for filtration and reuse of wash waters or aerobic unit effluent for toilet flushing in water-short areas.

A study is planned for demonstration of flow reduction devices and technology for 8 homes. Present water usage will be monitored prior to installation of modified plumbing fixtures. The program will include:

4 homes with flow control showers and shallow trap-dual cycle toilets

2 homes with flow control showers and with recycled (filtered) wash water reused in normal trap-dual cycle toilets

2 homes with shallow trap-dual cycle toilets and flow control showers, and with recycled (filtered) wash water used for lawn watering

PRESSURE SEWER SYSTEMS

Collection and treatment of wastewater at a central point should be utilized where feasible in lieu of individual home systems. However, it is not always practical to install conventional gravity sewer systems because of rough terrain and necessity for rock excavation. One possible solution is use of a pressure system, which is being studied under an R&D Grant at Grandview Lake, Indiana.

The Grandview Lake system will serve about 60 homes initially. Individual grinders and pumps will be installed at some homes. Septic tank effluent will be pumped directly without grinding at others. Three and one-fourth inch PVC pipe will be used for the pressure sewer with 1-inch connections from homes. Wastes will be treated by a combined anaerobic and aerobic lagoon. Lagoon effluent will be utilized for irrigation of a hay field.

The individual home pump and grinder unit was designed using commercially available home garbage grinders, pumps, and check valves. Unit cost without installation is $450 to $500.

VACUUM SEWER SYSTEMS

One of the most interesting developments in water closets and waste collection is the Liljendahl vacuum system. This system uses air rather than water as the major transport system for a vacuum toilet. A vacuum toilet requires only one-half gallon per flush compared to 4 to 6 gallons for conventional toilets. The vacuum system can also be utilized for waste collection from groups of homes in lieu of conventional gravity sewers.

The Sanivac Division of National Homes Corporation is marketing the Liljendahl vacuum system. This system has been used in Europe, the Bahamas, and in Latin America. A grant project is being developed for demonstration of the vacuum system in an area now served by septic tanks. Connections with house sewers will be made at lot lines. Another grant is planned for demonstration of the vacuum toilet system.

ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

MUNICIPAL POLLUTION CONTROL TECHNOLOGY
NON - SEWERED WASTES
PPB 1105

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

PPB - 1105 - NON-SEWERED MUNICIPAL WASTES

INTRODUCTION

A study on state-of-the-art of individual home waste treatment systems has been completed by Electric Boat Division of General Dynamics Corporation (Report 11050 FKE, "Flow Reduction and Treatment of Waste Water from Households," available September, 1970). In addition, an inhouse survey of proprietary equipment developed by industry but not now commercially available has been conducted.

SEPTIC TANK SYSTEMS

At the present time, there are approximately 15 to 17 million septic tanks in use. 1.4 million new systems have been installed since 1960. In many cases, operation of septic tank systems has not been satisfactory and has resulted in health hazards. This is due to poor soils which do not readily accept effluent. Lack of maintenance by homeowners is also a contributing problem. Regulatory agencies now require large lots in areas with low soil permeability. This has prohibited housing developments in many areas. However, in areas with high soil permeability and low population density, septic tanks usually perform satisfactorily and will continue to be an acceptable disposal method.

Septic tank costs are upwards from $120 and installation from $250. Tile field costs vary from $200 to $2400 depending on type of soil and regulatory requirements.

ACTIVATED SLUDGE PACKAGE PLANTS

Individual home aerobic package plants have been marketed since about 1955. It is estimated that there are 20,000 to 30,000 units now in operation. Initially, State health departments allowed discharge to natural waterways. Because of periodic discharge of suspended solids, subsurface disposal is now required by most regulatory agencies. Unit costs vary from about $800 to $1600 installed. If filtration and disinfection are required, costs are increased by $300 to $800.

DEVELOPMENT OF IMPROVED SYSTEMS

If an acceptable individual home treatment unit to replace septic tanks could be developed, a multimillion dollar market would exist. This potential market has induced industries to expend funds for R&D. From results of the inhouse survey, there are 5 companies that have conducted studies on unit development. In several cases, system components and total systems are now ready for field evaluation.

The basic problem in unit development is acceptable capital and operating costs. The Electric Boat study has concluded that use of advanced treatment processes, including distillation, reverse osmosis, electrodialysis, chemical treatment, and activated carbon adsorption, is not economically feasible at this time.

The basic problem area is solids disposal, as is the case for large-scale conventional treatment. Capital and maintenance costs for incineration are expected to be high. Another problem area is air pollution potential of incineration. Solids could be stored in the unit and disposed of similarly to sediment from septic tanks. However, this may not be aesthetically acceptable to all homeowners.

Fail-safe design is necessary. Homeowners tend to ignore operating and maintenance requirements for presently available package plants. There have been cases where power to units has been shut off. Policing of a large number of units by regulatory agencies is difficult. A possible solution to maintenance requirements is a permanent service contract with the manufacturer.

FUTURE STUDIES

In view of effort expended by industry on development of new equipment and approaches to individual home treatment, support of basic research on new systems or hardware development is not planned at this time. Planned studies include demonstration of commercially available hardware and modifications to improve treatment, and new proprietary equipment developed by industry. Studies on the relative absorption rates of effluent from package plants as compared to septic tanks will be conducted.

ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

VIROLOGY

PPB 1603 - 1706

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

Virology Section
Robert A. Taft Water Research Center
Cincinnati, Ohio
1603-1706

Introduction

Large quantities of viruses of human origin are present in sewage, sewage effluents, and in rivers and streams. Viruses of animal, plant and bacterial origin must also abound in these waters, but their presence is not as well documented, and their importance to the human animal is largely unknown.

The viruses of human origin are small in numbers compared with the numbers of bacteria that are excreted. Moreover, viruses do not multiply outside of living susceptible cells, and these numbers decrease even in that most nutritional environment constituted by domestic waste. The great importance of viruses in water, however, lies not just in their numbers, but in their great capacity to infect their hosts. The smallest amount of virus capable of infecting the most highly susceptible cells in cultures, our most sensitive indicators of infection, is usually capable of producing infection in man.

The clear capability of minimal quantities of viruses for producing infection in man is sufficient justification for seeking the total removal of viruses from any waters which man might consume. The permissible level for viruses in such waters should be none.

Epidemiology

Small amounts of viruses ejected into rivers and streams with partially treated wastewater become a potential hazard to downstream recreationalists and to those in downstream communities who must consume these waters. Even 19 PFU per 50 gallons of river water, an amount we have recovered with an inefficient technic even in cold months, constitutes a considerable hazard, for what this means in terms of the amounts of viruses that may enter the intakes of any community every day is readily calculable.

Unfortunately, small amounts of waterborne viruses may infect swimmers and consumers and not be readily detected by the effect they produce. This is so because small amounts of ingested viruses are likely to produce infection, but not disease. Infection is the state whereby the virus enters and multiplies within susceptible cells. It is a disease state with no overt signs. Overt disease exists when sufficient damage has been done to bring about systemic malfunctions. Thus, individuals infected with small amounts of virus may show no signs; yet, they may excrete large amounts of virus, their contacts may be infected with large amounts and recognizable illness may result. The spread of infection and disease in this fashion will appear to be by the personal contact route with no indication that the original source was water. The frustration of epidemiological studies intended to demonstrate a water source of transmission may be the result of using clinical illness and not index infection rates as criteria. In bathing water and similar studies, the disease rates in secondary contacts might well be a much better indicator of source than the disease rates in bathers themselves. This same principle may hold for bacterial infection and disease, for the main thrust of efforts in this area has always been in the direction of disease and not infection.

Recovery of Small Quantities of Viruses from Large Volumes of Water

Several years ago, we set a tentative standard for ourselves of less than 1 PFU of virus per 100 gallons of water. This standard was based on our assessment that detection of one PFU of virus in 100 gallons of water would be feasible within five years of that time, and not on any conviction that water with less than that amount of virus would be safe. It was our intent to raise our sights if developing technology allowed, and to lower them should a more modest limit need to be imposed upon us.

Clearly, the recovery of 1 PFU of virus from one-hundred gallons of water or more requires exquisite concentration procedures, and many are under study. It is not yet clear which of the several systems presently under investigation will prove the most efficient and utilitarian if, in fact, any one does become universal in all of the several applications for which such methodology is needed. Viruses must be recovered from waters of qualities ranging from raw sewage to completely renovated.

Except in the unusual situation where raw sewage or primary effluents need to be pasteurized or sterilized, only small volumes of such waters need to be tested for viruses because relatively large amounts of viruses are usually present. This is generally true of secondary effluents as well, and in these situations, effective technics, not adaptable to large volume efforts, are already available. The $Al(OH)_3$ adsorption procedure is reportedly capable of recovering 100% of several enteroviruses experimentally added to sewage effluents, but the method leaves most of the large reoviruses and adenoviruses behind. England (16030 DWW) recently reported efficient recovery of experimentally added reoviruses and adenoviruses from effluents by precipitation with protamine sulfate which leaves most of the smaller picornaviruses behind. England uses both methods for maximum recovery of all of these viruses. Protamine precipitation of the larger viruses from $Al(OH)_3$-adsorbed effluents is an important approach today to the effective recovery of viruses from heavily contaminated waters.

The phase separation technic suffers some disadvantage from an overnight time requirement for completion. It has also been reported recently that the method is not efficient with all viruses.

Both the $Al(OH)_3$-protamine sulfate and the phase separation procedures are limited by the volumes they can accommodate, this the result of the large quantities of chemicals required for each unit volume of water tested. When only a few liters or gallons need to be tested, these technics may be considered. When a hundred gallons or more must be tested, other methods must be looked to.

To accommodate large volumes of water, a filtration system seems the best approach. This technic consists of filtering water through 0.45 μ cellulose nitrate membrane filters to which viruses adsorb and from which they can be eluted. For some time now, we have consistently obtained quantitative recovery of enteroviruses and about 80% recovery of reovirus 1. Most of our studies were done with 1-liter samples, but we achieved complete recovery of viruses from 25 gallon quantities as well. With larger quantities of water, recovery has so far been less efficient. In most of these experiments, less than 100 PFU of virus were added to the total volume studied. Most of these experiments were done in distilled water, but several were done in tap water from which we could not always recover viruses quantitatively. As others have reported, certain substances, presumedly organics, apparently can react with the adsorptive sites on the membranes and make them unavailable to the virus. Thus, our immediate goal is to apply the technic to renovated and other clean waters, but it may be necessary to pretreat even such relatively clean waters to remove interfering substances before the membrane filter method can be effectively used for quantitative recovery of viruses. Whether waters of poorer quality can be sufficiently purified without removing or destroying viruses so that such waters can be tested with this technic is still conjecture. However, there is another filtration approach currently in a state of reincarnation that offers promise for quantitative recovery of viruses from water--the ion exchange resin, more voguishly, the insoluble polyelectrolyte. This method consists of filtering water through two Millipore AP 20 fiberglass prefilters between which a Monsanto insoluble polyelectrolyte designated PE 60 is sandwiched. The virus is eluted with 10% fetal calf serum in borate saline at pH 9.0. In our hands, when small amounts of viruses in 1-liter volumes of distilled water were passed through such filters, relatively poor recoveries resulted. Poliovirus 1 recovery in experiments sometimes ranged over 80%, but echovirus 7 recoveries were sometimes somewhat lower than 30% and reovirus 1 recoveries were sometimes lower than 20%. The extent to which the ion exchange resin is affected by water quality is not clear, but apparently, it is less affected than the celluiose nitrate filter.

Nonetheless, we have repeatedly used the technic for virus recovery studies from 50-gallon samples of river water, and repeatedly obtained recoveries. As much as 19 PFU of virus have been recovered from samples taken long distances from outfalls along a large fast-flowing river during the winter months. Thus, despite its low and erratic efficiency at present, the technic appears to be the most sensitive presently available. Since the numbers of different ion exchange resins that can be produced is vast, these substances clearly warrant the reincarnation they now experience.

Other technics including osmotic ultrafiltration and electro-osmosis are also under study, but it is not yet clear what their final contribution to the developing technology will be. Nor is it clear at present whether we will eventually achieve a universal recovery system that can be utilized efficiently with waters of all qualities, or whether we will have to tailor the recovery system to the water under study.

Gerald Berg, Ph.D.
July 7, 1970

ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

DISSOLVED NUTRIENT REMOVAL FROM WASTEWATER

PPB 1701

Division of Process Research & Development
Federal Water Quality Administration
U.S. Department of the Interior

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NITROGEN REMOVAL - GENERAL

Municipal wastewaters have nitrogen contents in the 15-25 mg/l range in untreated and primary settled wastes; the nitrogen is divided between organic compounds, which are mostly insoluble, and ammonia. In general, we can depend on conventional biological processes to transform almost all nitrogenous components in wastewater into ammonia and biological sludge. Once this has been accomplished, we can design systems to remove ammonia by air-stripping. Ammonia stripping at high pH in cooling towers following lime treatment is effective but cannot be used during freezing weather and may suffer from serious scale problems.

Under favorable conditions, biological processes may also oxidize ammonia to nitrates by a two-step sequence called nitrification. It would be beneficial if waste treatment plants were required to produce nitrified effluent. Ammonia nitrogen in effluents has several undesirable features:

(1) Ammonia consumes dissolved oxygen in the receiving water;

(2) Ammonia reacts with chlorine to form chloramines which are less effective disinfectants than free chlorine;

(3) Ammonia is toxic to fish life;

(4) Ammonia is corrosive to copper fittings;

(5) Ammonia increases the chlorine demand at waterworks downstream.

A nitrified effluent, free of substantial concentrations of ammonia, offers several advantages:

(1) Nitrates will provide oxygen to sludge beds and prevent the formation of septic odors;

(2) Nitrified effluents are more effectively and efficiently disinfected by chlorine treatment;

(3) A nitrified effluent contains less soluble organic matter than the same effluent before nitrification.

A nitrified effluent is far preferable to one containing substantial ammonia. However, ammonia and nitrate are interchangeable nitrogenous nutrients for green plants and algae, as well as bacteria. If the nitrate level is too high and is helping to stimulate undesirable aquatic growths, the effluent can be further treated by biological action to convert the nitrates to nitrogen gas. This process is called denitrification. The best developed method at this time for control of nitrogen compounds is biological oxidation to nitrates followed by denitrification with the aid of methanol.

Selective ion exchange of ammonia with lime regeneration may be practical but the process is still in the pilot stage. Several other processes are being studied including selective ion exchange of nitrate and chlorination of ammonia to liberate nitrogen gas.

NITROGEN REMOVAL BY BIOLOGICAL SUSPENDED GROWTH REACTORS

Success in providing a high efficiency for nitrogen removal by biological denitrification requires that the biological transformation of ammonia nitrogen to nitrate nitrogen be under good process control. Any reduced nitrogen compounds introduced into the denitrification stage will pass through the process unaltered and impair overall nitrogen removal efficiency.

Complex factors are involved in maintaining nitrification with a conventional activated sludge system. If nitrification occurs at all, it may be due only to an unintentional accident of design. A three sludge variation of the activated sludge process, developed at the Robert A. Taft Water Research Center, greatly simplifies the process control problems associated with maintaining nitrification.

The three sludge system allows management of the separate biological transformations which are necessary for successful denitrification. The three sludge systems are staged in sequence, with flow passing from one stage to the next. The first stage is a high-rate sludge system, the second stage a nitrification sludge system, and the third a denitrification sludge system. The high-rate system handles the bulk of the carbonaceous removal and at this station the waste activated sludge is removed. Thus, the nitrification stage receives a predominantly ammonia nitrogen feed and an enriched culture develops because each sludge system has its own sludge recycle. This process design also has other desirable features. The high rate system protects subsequent nitrification stages from toxic chemicals. Since this is a staged system there can be no direct short circuiting of materials from the influent to the effluent. Temperature effects on the enriched culture of the nitrification stage are not as extreme as with a single sludge system which contains only a marginal population of nitrifying organisms.

Once controlled nitrification has been established, the biological denitrification process can be optimized. The nitrified effluent flows to a slowly stirred anaerobic reactor where methyl alcohol is added in proportion to the nitrate nitrogen concentration. The organisms in this stage use the oxygen component of the nitrate radical to oxidize the organic carbon of methyl alcohol. The end products of this metabolism are elemental inert nitrogen gas and carbon dioxide, which are liberated to the atmosphere.

The stage approach to nitrification has been investigated in work at the Robert A. Taft Water Research Center (1) and in large pilot plant operations at the University of Notre Dame (2) and Manassas, Virginia (3). The process has also been evaluated on a 1 mgd scale at Hazel Crest, Illinois. A summation of these studies show that biological denitrification is a controllable process if the reaction is forced with an organic supplement, such as methyl alcohol. Total nitrogen in an effluent can be reliably reduced to about 2 mg/l. The cost of the methyl alcohol for 20 mg/l of nitrate nitrogen is estimated to be about 2c/1000 gallons treated. For more information contact:

Mr. E. F. Barth
U. S. Dept. of the Interior, FWQA
4676 Columbia Parkway
Cincinnati, Ohio 45226

Phone: 513-871-1820

References:

(1) Barth, E. F., et al., "Chemical-Biological Control of Nitrogen and Phosphorus in Wastewater Effluent," Jour. Water Pollution Control Federation, December 1968.

(2) Echelberger, W. F. and Tenny, M. W., "Control of Organic and Eutrophying Pollutants by Combined Chemical and Biological Wastewater Treatment," Division of Water, Air and Waste Chemistry, American Chemical Society, Minneapolis, Minnesota, 1969.

(3) Mulbarger, M. C., et al., "Manassas, Virginia, Adds Nutrient Removal to Waste Treatment," Water and Wastes Engineering, April 1969.

NITROGEN REMOVAL FROM WASTEWATERS BY COLUMN REACTORS

Columnar nitrate reduction represents a second alternative to the suspended growth systems as a means of biochemically reducing the nitrate ion to elemental nitrogen. In a packed column, the cell residence time of the surface bound slime is much greater than the hydraulic detention time. This, combined with a large contact surface and short diffusion distances afforded by small media such as sand, provides an efficient system for rapid denitrification of an applied feed.

Work at the FWQA Lebanon, Ohio Pilot Plant (J. M. Smith, 1970, Unpublished) has shown that the smaller media systems (sand to 3/4 inch diameter stone) are effective when operated downflow at surface loading rates of 7.0 gpm/ft and at actual contact times of 50 to 30 minutes. Daily backwashing is required to relieve pressure drop due to the accumulation of suspended solids in the upper portion of the column. The denitrifying slime is firmly attached to the media surface, and is not removed during the backwash operation. Greater than 90 percent nitrate reduction can be achieved within these columns at contact times of 10 minutes for sand and 30 minutes for the 3/4 inch stone. The effluent normally contains less than 2.0 mg/l of nitrate nitrogen with effluent turbidities less than 3 JTU, indicating little solids contribution from the attached organisms.

Larger media varying in size from 1 inch to 2 inch aggregate have been successfully employed to denitrify agriculture subsurface drainage at Firebaugh, California (Tamblyn, T.A., and Sword, B.R., "The Anaerobic Filter for the Denitrification of Agricultural Subsurface Drainage," 24th Purdue Industrial Waste Conference, 1969.) The larger media permits upflow operation without backwashing at the expense of longer contact times and increased effluent suspended solids. Nitrate reduction of greater than 90 percent were achieved in contact times of 1 hour for the 1 inch aggregate and 2 hours for the 2 inch aggregate at temperatures above 12°C. The 2 inch columns have been operated continuously for over six months on agriculture subsurface drainage without the loss of efficiency or solids accumulation.

As with suspended growth denitrification, methyl alcohol is used as the supplemental organic carbon source of choice for columnar denitrification because of its low cost, biodegradability and ease of handling. Approximately 3 mg of methyl alcohol are required per mg of nitrate nitrogen removed including the requirement for deoxygenating the nitrified feed. The chemical cost for removing 20 mg/l of nitrate nitrogen in the presence of 5 mg/l of dissolved oxygen is estimates to be about 2¢/1000 gallons treated. For additional information, contact:

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AMMONIA NITROGEN REMOVAL BY STRIPPING WITH AIR

Ammonia can be removed from a wastewater effluent by raising the pH to convert ammonium ion to dissolved ammonia and then contacting the effluent with a sufficient quantity of ammonia-free air. This physical process is called desorption or, more commonly, "stripping."

If the contacting is done in a packed tower, the pressure drop across the tower is about 1.0 psi or 28 inches of water. Since the volume of air required per unit volume of wastewater effluent is very high, about 400 cubic feet per gallon in a countercurrent operation, the cost for power to overcome even this relatively low pressure drop is prohibitive.

The problem of high power cost was solved by investigators at the South Lake Tahoe Public Utility District (Slechta, A. F. and Culp, G. L., "Water Reclamation Studies at the South Tahoe Public Utility District," Jour. Water Pollution Control Federation, May 1967).who used a slat-filled tower such as is used for cooling water to contact water and air. The pressure drop across such a device is very low, about 1/2 inch of water, so power costs are reduced to reasonable levels. Removal efficiencies as high as 90 percent were obtained in a 24-foot high tower in which wastewater effluent and gas were contacted in a nearly countercurrent fashion. On the basis of this experience, a full-scale stripping tower was constructed at South Lake Tahoe. The tower was designed to remove 90 percent of the ammonia from 3-1/2 MGD of Tahoe's removated wastewater. The air flow is not countercurrent to the liquid but flows across the tower (cross-flow), while the wastewater drips downward through the packing.

Initial operation of Tahoe's stripping tower was in the winter and immediately revealed a limitation of ammonia stripping. When air temperature fell below 0°C,freezing of water occurred at the air inlets, making the tower inoperable. Also, since ammonia solubility is higher in cold water than in warmer water, more air is required to remove it (800 cubic feet per gallon at 0°C). The Tahoe tower was designed for 400 cubic feet per gallon; therefore, removal was much lower than 90%.

Another problem which developed at Tahoe is the formation of scale in the tower. The scale is chiefly calcium carbonate. It forms because the previously lime-treated effluent is supersaturated with respect to calcium carbonate. In the case of the tower at Tahoe, the sludge can be flushed from the tower except from inaccessible areas which cannot be reached with a water jet. A pilot scale ammonia stripping tower at FWQA's Blue Plains, Washington, D. C. Pilot Plant, has had similar scaling problems, except the scale is hard and adheres to the tower fill. The causes of the differences in the nature and amount of scale in various locations has not been resolved. Studies are in progress to see if the scale can be prevented from forming, or if it can be made nonadherent.

The cost of ammonia stripping has been estimated for the South Lake Tahoe facility to be about 2.9¢ per 1000 gallons of wastewater treated. This does not include the cost of the lime and facilities to raise the pH to about 11. These costs have been charged to phosphorus removal because this is the direct objective of the lime addition. If 90% removal of ammonia nitrogen is required even in cold weather, these costs should be increased by about 50% to provide for a higher air-to-water ratio.

Ammonia stripping is feasible when the temperature is above freezing but there is danger of serious fouling by scale. The best approach for minimizing scale and its effects appears to be to use a pH of about 10.5, countercurrent operation rather than cross-flow, and an open fill to allow for easy flushing of accumulated solids. For more information contact:

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NITROGEN REMOVAL BY CHEMICAL METHODS

A. Removal of Ammonia by Selective Ion Exchange

Conventional water softening ion exchange resins which are selective for calcium and magnesium do a relatively poor job of removing ammonium from dilute solutions. Total deionization by mixed bed ion exchange resins will, of course, remove ammonium ions along with other cations but this process is too costly for wastewater treatment.

Certain zeolites show unusual selectivity for the ammonium ion. A number of these have been investigated by the Atomic Energy Commission because they also show selectivity for cesium and potassium ions. A demonstration project at the Battelle Memorial Institute - Pacific Northwest (Hanford) Laboratories, 1969 (Mercer, B. W., et al., "Ammonia Removal from Secondary Effluents by Selective Ion Exchange," Jour. Water Pollution Control Federation, Research Supplement, February, 1970) showed that certain zeolites, including the naturally occurring mineral clinoptilolite, had a high selectivity for ammonium in natural and wastewaters. A trailer mounted demonstration plant with a capacity of 100,000 gallons per day was built as a cooperative demonstration project between the FWQA and Battelle-Northwest. This trailer is now operated under contract to the FWQA to demonstrate selective ion exchange removal of ammonium ions from solution.

Clarified secondary effluent is passed downward through columns containing clinoptilolite. When a column becomes loaded with ammonia, it is regenerated with limewater containing sodium chloride to speed up the rate of regeneration. The high pH of the limewater converts the ammonium ion to unionized ammonia gas in solution. The ammonia laden limewater is then pumped through a packed column through which heated air is blown to remove the ammonia.

Pilot studies at Battelle-Northwest indicated a cost approaching 10¢/1000 gallons for their Zeolite method. At Lake Tahoe, where winters are severe and the area is not readily accessible for chemical deliveries, a cost of 15¢/1000 gallons has been estimated for a 7-1/2 mgd plant. More reliable cost estimates will be available at the conclusion of the present contract with Battelle-Northwest.

B. Ion Exchange for Nitrate Removal

Several attempts have been made to develop selective ion-exchange processes for nitrate removal. Dow Chemical Company is presently under contract to FWQA (Contract No. 14-12-808) to develop a nitrate removal process based on the use of a porous solid absorbent containing a nitrate-selective water-immiscible extractant. The process has the advantages of liquid ion-exchange technology and the ease of operation of the granular bed resin systems.

Selective nitrate removal by ion exchange will not be feasible until new resins are synthesized with a high selectivity for nitrate over other anions present in the water. In addition, a suitable process for treating the nitrate laden regenerants must be developed.

C. Chlorination of Ammonia

Ammonia can be oxidized to nitrogen gas by chlorinating to the breakpoint with either chlorine gas or sodium hypochlorite. Four moles of chlorine or hypochlorite per mole of nitrogen gas liberated are required. Hypochlorite is more expensive than chlorine gas, but it is much safer to transport and handle.

Breakpoint chlorination, of course, also disinfects the wastewater as well as oxidizing ammonia. However, the addition of 200-300 mg/l of chloride ion would not be acceptable for many inland waters.

Assuming 20 parts of ammonia nitrogen in a secondary effluent, 200 parts of chlorine would be required for breakpoint chlorination. This is equivalent to 1.5 lbs of chlorine per 1000 gallons or about 6¢/1000 gallons. To this must be added the cost of handling the corrosive hydrochloric acid produced. Sodium hypochlorite may cost twice as much as chlorine but associated costs are greatly reduced.

For more information on ammonia removal by selective ion exchange or breakpoint chlorination, contact:

Dr. R. B. Dean
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4676 Columbia Parkway
Cincinnati, Ohio 45226
Phone: 513-871-1820

For additional information on selective nitrate removal, contact Mr. R. A. Dobbs at the same address.

PHOSPHORUS REMOVAL - GENERAL

Phosphorus is considered by many investigators to be the key nutrient in breaking the eutrophication cycle, However, conventional secondary plants are not efficient in phosphorus removal. Phosphorus enters a plant in the highest oxidized form. But, no common biological systems reduce phosphorus; therefore, it cannot be liberated in a gaseous form as nitrogen, carbon, and sulfur are. Removal by biological means, then, is limited to cell metabolic needs and whatever excess phosphorus can be encouraged to be taken by and stored by the cells. The quantity stored above the 1% required for maximum growth is usually classified as "luxury uptake."

A few plants have reported efficient phosphorus uptake on a sustained basis, including the San Antonio Rilling Plant and the Baltimore, Maryland Plant. These results cannot be readily duplicated at other plants by manipulation of operating conditions. We have not learned enough about the phenomenon to take advantage of it. The removal of phosphorus by biological synthesis and "luxury uptake" is not a controllable process at this time.

If we are to reliably remove phosphorus from wastewaters on a sustained basis, we must choose the chemical or the chemical-biological methods. Strict chemical methods precipitate phosphorus either in the primary settler or in a tertiary clarifier. The chemical-biological method employs direct chemical dosing to the aerator of an activated sludge plant. The chemically-bound precipitated phosphorus is removed with the sludge and is not resolubilized during sludge disposal unless the pH is substantially lowered. Effluent phosphorus concentrations of 1-2 mg/l as P can be regularly achieved if the precipitation is accomplished in the primary or secondary portions of the plant. Tertiary lime clarification followed by filtration will lower the concentration to less than 0.5 mg/l.

BIOLOGICAL PHOSPHORUS REMOVAL

The literature indicates that several factors exert an influence on biological phosphorus removal. The rate of aeration and the aeration time have been indicated by most investigators as the most important criteria, the rate of air supply probably being the more critical of the two. Aeration rates in the order of 3 to 7 cfm/gal and detention times of 4 to 6 hours appear to be desirable.

There is some disagreement in the literature with respect to optimum concentration of mixed liquor suspended solids (MLSS). Apparently, increased uptake has been attained at both low and high MLSS from 500 mg/l up to 4300 mg/l. At the San Antonio, Texas treatment plants, the optimum appeared to be 1000 mg/l or slightly higher (1). It was also found that the maximum overall phosphorus removal occurred at organic loadings of 45 to 55 pounds of BOD/day/100 pounds of MLSS under aeration.

It also appears essential from the literature that a dissolved oxygen (DO) level of at least 2 mg/l should be maintained in the last half of the aeration tank to insure that phosphorus will not be released in the secondary clarifier. It is possible that a still higher DO level of 3 to 5 mg/l may be advantageous to maintain a minimum DO concentration of 1.5 mg/l in the sludge until it is through the secondary clarifier. Phosphorus leakage or resolubilization will occur in the secondary clarifier when the sludge consumes available dissolved oxygen. It has been suggested that solids detention time in final clarifiers should be less than 30 minutes.

These key design criteria and operational parameters have not been sufficiently isolated and identified to effectively predict and implement controlled phosphorus removal by the solely metabolic mechanism. As more data have been collected, an alternative chemical explanation has been advanced (2). Simply stated this theory indicates, especially in hard water areas, that phosphorus can be precipitated within the biological floc as calcium phosphate at the end of the aeration period, where carbon dioxide is scrubbed from the water by aeration and a substantial increase in pH occurs. This amount of precipitated calcium phosphate and the precipitation of additional phosphorus by traces of iron, aluminum, and magnesium normally present in wastewater would produce an efficient overall removal.

The calcium phosphate theory has been tested at several treatment plants with erratic results. Operating a segment of the Hyperion, California Plant according to the guidelines outlined by the theory has greatly increased the efficiency of phosphorus removal. At Baltimore, Maryland where efficient phosphorus removal occurs routinely, observations show no major increase in pH during operation. Studies at Texas City, Texas where attempts were made to deliberately force calcium phosphate precipitation by the addition of 200 mg/l of lime to the aerator have not shown efficient removal.

The preliminary data reported from these full-scale treatment plants are still not complete or detailed enough at this date to confirm either the metabolic or calcium phosphate precipitation theory. For further information contact:

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U. S. Dept. of the Interior, FWQA
4676 Columbia Parkway
Cincinnati, Ohio 45226
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PHOSPHORUS REMOVAL BY MINERAL ADDITION TO THE PRIMARY OR SECONDARY

Mineral addition is out of the research stage and into the application stage. Field experience on full-scale and large demonstration pilot plants shows that ferrous, ferric, and aluminum salts can be equally effective as phosphorus precipitants in wastewater. Plants can accomplish 80 to 90 percent phosphorus removal with a minor investment in capital equipment for chemical storage tanks, chemical pumps, and control equipment (Barth, E. F. and Ettinger, M. B., "Mineral Controlled Phosphorus Removal in the Activated Sludge Process," Jour. Water Pollution Control Federation, August, 1967).

For trickling filter plants, the chemical precipitation should be accomplished in the primary tank. Direct dosing of chemicals to the trickling filter has not proven highly effective. A small dose of polymer is needed to flocculate and settle the phosphorus which is insolubilized by the mineral addition. Subsequent passage through the trickling filter to satisfy metabolic needs serves as a polishing step. Dow Chemical has conducted several studies of iron-polymer precipitation in the primary at Midland, Lake Odessa, Grayling, and Benton Harbor, Michigan. FWQA sponsored projects include Grand Rapids, Michigan (45 mgd) and Richardson, Texas (1.5 mgd). For further information contact:

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With an activated sludge plant, it makes very little difference where the point of addition of the metal ion is. Efficient removals have been obtained when dosing raw wastewater before primary settling, after primary settling, in the aeration tank, or near the mixed liquor exit point. Physical constraints of a particular plant may favor one point of addition over another. However, the key factor in this approach is that no matter where the metal ion insolubilizes the phosphorus, the overall plant efficiency is dependent upon the ability of the biological floc to collect these dispersed precipitates and remove them from the final plant effluent. Polymer addition in the primary is not necessary for an activated sludge plant as the naturally occurring polymeric materials in the mixed liquor will serve the same purpose. FWQA sponsored projects of phosphorus precipitation in an activated sludge plant include Penn State University (2 mgd), Texas City, Texas (0.75 mgd), University of Notre Dame (50,000 gpd), Manassas, Virginia (1 mgd), Xenia, Ohio (1 mgd), and Detroit, Michigan (7,000 gpd). For more Information contact:

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Dosages of 1.5 to 2.0, on a molar basis, of metal ion to phosphorus can produce effluents with a residual total phosphorus of 1 milligram per liter or less consistently on full-scale application. As is true with other parameters such as BOD, COD, and suspended solids, if very low residuals are desired, filtration of the effluent would be required. If commercial aluminum and iron minerals are used, the chemical cost will vary from 2-5c/1000 gallons, depending on the phosphorus concentration and the chemical employed. If waste pickle liquor is available for the cost of trucking only, the chemical cost may be as low as 0.5¢/1000 gallons.

The mineral addition process can be superimposed on the three sludge biological denitrification process to achieve efficient phosphorus, nitrogen, carbon, and suspended solids removals in one integrated treatment sequence (Barth, E. F., et al., "Chemical-Biological Control of Nitrogen and Phosphorus in Wastewater Effluent," Jour. Water Pollution Control Federation, December, 1968). In this process, the recommended major point of mineral addition is to the high-rate unit with a small polish dose to the denitrification unit to precipitate residual phosphorus.

LIME PRECIPITATION OF PHOSPHORUS

Dorr-Oliver's Phosphate Extraction Process (PEP) is the only commercially advertised lime precipitation process for use in the primary. In this process, a solids contact type reactor-clarifier is used instead of a conventional primary settler. Clarifier underflow solids are recycled to the raw sewage to maintain 500-2,000 mg/l of suspended solids in the reactor. The objective of the PEP process is to achieve 80% phosphorus removal in the primary, depending on the subsequent activated sludge step to increase overall removal to 90% or greater. For further information contact:

Mr. O. E. Albertson
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Currently, lime precipitation is also being considered as the first step in a chemical-physical treatment sequence for raw wastewater that does not include a biological unit. Subsequent units in the sequence include lime recovery, filtration, carbon adsorption and possibly ammonia stripping.

The above sequence is similar to the tertiary sequence demonstrated for several years at Lake Tahoe's 7.5 mgd Water Reclamation Plant. The Lake Tahoe Plant utilizes secondary effluent as feed water. Phosphorus removal costs at Tahoe vary monthly from 6-8¢/1000 gal. including amortization, operating costs, and recalcination.

For additional information on chemical-physical treatment of raw wastewater, contact:

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For additional information relative to Lake Tahoe's operations, contact Dr. R. B Dean at the same address.

ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

DISSOLVED REFRACTORY ORGANICS

PPB 1702

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

DISSOLVED REFRACTORY ORGANICS

PPB 1702

PROCESSES FOR ORGANICS REMOVAL

I. Introduction

Most liquid wastes, both domestic and industrial, contain a complement of organics which must be removed or altered before discharge. The classical approach and the method now most widely used has been biological oxidation. Decades of research have produced a great variety of processes, all dependent on biological activity, to consume organics for energy and for cell protoplasm. Biological oxidation has limitations: some organics are not degradeable, toxic materials must be avoided and low temperatures slow biological activity. Recognition of these limitations plus the need to produce increasingly higher quality effluents for discharge or for reuse, led AWT to search for alternatives to biological treatment.

Several processes for removal of organics from both domestic and industrial waste streams are in varying stages of development. These are:

1. Granular activated carbon
2. Powdered activated carbon
3. Adsorbent resins
4. Oxidation processes

II. Granular Activated Carbon

Activated carbon is an adsorbent medium characterized by an extensive system of internal pores which provide it with a very large surface area per unit of weight. This large area plus the variety of functional groups (acidic, basic, oxygenated, etc.) attached to the surface give activated carbon a significant adsorptive capacity for most dissolved organics in wastewater. The carbon, when exhausted, can be reused after regeneration by heating to high temperature (ca 1700°F).

The method of application is primarily determined by the particle size of the carbon to be used. Granular carbon, in the mesh size range from 8 x 30 to 40 x 60, is generally contacted with the wastewater in a fixed or fluidized bed of carbon. Originally, carbon adsorption was considered as a tertiary treatment to supplement biological processes to produce a high quality product of reuseable quality. More recently, the main thrust of research has shifted from the treatment of biological secondary effluent to treatment of clarified raw sewage. Success in the latter effort will provide the sanitary engineer an alternative to biological treatment.

One of the first large-scale applications of granular carbon to wastewater treatment was the South Tahoe Wastewater Reclamation Plant. This 7.5 mgd granular activated carbon plant treats secondary effluent after clarification by lime and mixed media filters. The carbon effectively reduces an influent BOD from 5-20 mg/l to 2-5 mg/l; COD from 20-30 mg/l to 2-10 mg/l; and color from 20-50 to less than 5 units. The average dosage of carbon to accomplish this treatment has been 300 lb/million gallons of treated wastewater.

Large-scale studies at Pomona have substantially confirmed the results obtained at Tahoe. Carbon dosage, however, was found to average about 350 lbs/million gallons. Here, too, effluent quality has been good. Total COD was reduced from 47 mg/l to 9.5 mg/l; color from 30 units to 3 units and turbidity from 10 JTU to 1.6. Significantly the CCE, which has been used as a measure of water quality for drinking water supplies, was 0.014 mg/l, substantially below the recommended 0.2 mg/l.

These two large-scale studies plus bench investigations firmly established that activated carbon can produce effluents with low organic contents and at a cost that is reasonable. To make the process economic it was recognized very early that multiple use of the carbon, in contrast to the single use practiced in water treatment, was necessary. Current regeneration techniques using temperatures of 1600-1700°F plus steam have been able to recover 92-95% of the carbon. Some losses, both physical and chemical, do occur during regeneration. Attempts to regenerate carbon in situ with chemical oxidants or caustic washes have not been successful.

The manner in which the carbon is contacted with the wastewater has been the subject of considerable investigation. The wastewater can be upflow or downflow; the carbon can be static or moved continuously or in slugs; or a fluidized bed can be used. In most of these applications pressure has been used to maintain flows. Simple gravity flow contactors (using lower flow rates) have been suggested as economic. Recent estimates by Swindell-Dressler show that the gravity flow system is less expensive by about 2¢/1000 gallons in spite of the smaller flow rate. Flow rates in pressure systems have ranged 6-10 gpm/ft^2 while gravity flow will range 2-4 gpm/ft^2.

The most thorough estimate of the cost of treating secondary effluent by carbon adsorption was prepared by Swindell-Dressler. Various systems were subjected to side-by-side economic analyses, using data then available in the literature. Total costs have ranged from as little as 8.5¢/1000 gallons for the gravity system to as much as 12.5¢/1000 gallons for a 10 mgd plant.

These studies and others have clearly established that activated carbon can produce good quality effluents from secondary effluent at some reasonable and predictable cost.

A more recent concept in the use of activated carbon is replacement of the biological secondary treatment process in conventional treatment. The process sequence consists of chemical clarification of raw sewage by either organic flocculants or by metal coagulants, when phosphate removal is desired, followed by carbon adsorption. To date, technical feasibility has been demonstrated only at small scale, but full-scale application will be demonstrated within the next two years.

Some impressive information has already been developed on this process which could replace biological treatment by a purely physical-chemical process. Calgon's studies of the treatment sequence (clarification-carbon) has shown the following removals are obtainable when contact time with the carbon is 24 minutes; suspended solids 93%; BOD 93%; COD 81% and TOC 75%. When metal coagulants are used in the clarification step, phosphate removals in excess of 90% can be obtained.

Pilot scale investigations at the Lebanon Pilot Plant of AWTRL have shown that lime clarification followed by carbon adsorption of primary effluent can consistently produce an effluent equal or better in quality than secondary biological treatment. Over five million gallons of primary effluent were processed to produce an average effluent product containing 10 mg/l TOC and BOD with a range of 2-23 mg/l. Effluent turbidity averaged less than 2 JTU end phosphate removals were consistently 90% or better.

Some advantages that can be cited for a physical-chemical process are:

1. Substantially less land would be required. Calgon claims as little as 1/10.

2. Capital costs for conventional plants may be 30-40% greater than that for [illegible] P-C plant.

3. P-C process should be less influenced by shock loads, low temperature and by substances which would be toxic to a biological system.

4. The plant should be easy to operate and could be readily adjusted to produce a ranging quality of effluent as desired.

5. Odor problems should be minimal.

6. Significantly, much less sludge will need to be handled. For example, a conventional 10 mgd activated sludge plant will produce about 150,000 gpd of sludge, about 70% of which, or 105,000 gpd, is secondary sludge. The P-C plant could very well reduce the volume to about one-half of the total, depending on the flocculant used, and this sludge should be readily filterable.

A major disadvantage of the P-C process is that ammonia nitrogen will be unaffected. Substantial reductions of organic nitrogen can be expected through solids removal both by the clarification step as well as by the filtering function of the carbon beds.

The plant which will probably be the first to demonstrate the P-C process sequence is located at Rocky River, Ohio. While the original process envisions polymer flocculation, phosphate removal and clarification is being studied for possible use. The carbon adsorption plant will consist of eight pressure contactors, 25 feet high (15 feet of carbon bed) and 16 feet in diameter, and will process a peak flow of 20 mgd (nominal flow of 10 mgd). Flow rate will be 4.3 gpm/ft^2 with a peak rate of 8.6 gpm/ft^2. Carbon will be thermally regenerated at an anticipated rate of 300-500 lbs/day/million gallons. Loss on regeneration is expected to be no more than 5%. Effluent quality objectives are 15 mg/l BOD and 10 mg/l suspended solids, but actual quality may exceed these.

Another plant at Painesville, Ohio, will be designed for a flow of 5.0 mgd, part of which (up to one-half) consists of oil and chemical wastes. Fluctuations of pH from 2-11 and the presence of high concentrations of phenol and chlorine would make biological treatment difficult if not impossible.

Preliminary studies have shown that the wastewater can be effectively clarified (and phosphate precipitated) by ferric chloride. Initial plans call for clarification, roughing sand filters and gravity-flow carbon contactors. The latter will be 15 feet deep, containing 8 x 30 mesh carbon in columns operated in parallel at 2 gpm/ft^2. Effluent quality objectives are, BOD, 20 mg/l; COD, 30 mg/l; phosphates 80% removal and suspended solids 10 mg/l.

Status Summary

The technical feasibility of adsorption of organics by activated carbon has been well established. Regeneration of exhausted granular carbon can be considered to be operational. It remains for the two P-C demonstration plants discussed above to provide operational and cost information. If cost of P-C treatment is comparable to conventional biological secondary and for comparable effluent quality, then increasing numbers of these plants will be used. Reliability of the effluent quality, the smaller land requirements, the freedom from toxic influences, the lack of odor nuisance in areas of population, are some of the reasons why P-C plants will find increasing use.

III. Powdered Activated Carbon

Powdered carbon has developed into a rival of granular carbon. Its finer grain size increases the kinetics of adsorption such that 90% of its adsorption equilibrium is attained in less than 10 minutes. Powdered carbon is dosed in slurry form, after which it is separated by sedimentation following polymer flocculation. Other methods of separation are being investigated. Powdered carbon has the advantage over granular in that its cost is about 1/3 as great. Unit cost and the possibility to control the dosage applied are two of the advantages over granular.

Powdered carbon can be applied to either primary or secondary effluent and is being tested on both feeds. Determination of the technical and economic feasibility must await the results of contracts with Eimco Corp. and Infilco. In contrast to granular carbon regeneration, recovery of spent powdered carbon has been accomplished only in small prototype furnaces. Larger scale regeneration will have to be done before the powdered carbon process is a practical alternative to granular carbon.

IV. Other Methods for Organic Removal

At the present time, powdered and granular carbon provide the reagents of choice for removal of organics. Other methods, however, are being investigated as alternatives to carbon or for specialized applications. Adsorbent synthetic resins are available and newer ones are being developed which have the ability to sorb organics without any substantial inorganic exchange capacity. At this point of development, sorbent resins are not likely to replace carbon but the search for better ones is continuing.

A variety of chemical oxidation methods have been investigated such as chlorine catalyzed by U-V light, metal catalyzed photo-oxidation and ozone. Of these, only ozone appears to be promising. Technical feasibility was established in the laboratory by Airco, Inc., which is currently constructing a 50,000 gpd plant to establish economic feasibility. Because of the cost of ozone itself and the rather large doses, up to 100 mg/l, required for oxidation, application is likely to be limited to treatment of low organic content feeds, such as carbon effluents which need further organic reduction. A valuable benefit of ozonation is its disinfection of the waste stream.

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Sources of Information

Much of the research on processes for removal of organics from wastewaters is conducted at or out of the Advanced Waste Treatment Research Laboratory. The address of the laboratory and the principal investigators are given below:

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ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

JULY 1, 1970

SUSPENDED AND COLLOIDAL SOLIDS REMOVAL

PPB 1703

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

SUSPENDED AND COLLOIDAL SOLIDS REMOVAL

PPB 1703

PROCESSES FOR SOLIDS REMOVAL

I. Introduction

Removal of suspended or colloidal solids from domestic and industrial wastewater is of major importance in any treatment system. Evidence of its importance is the great variety of methods and devices which have been developed for this task. This brief review can only discuss the more widely used processes and describe the newer, more promising techniques now being developed. The subject material can be considered in two parts; the physical aspects which relate to the equipment and the physical methods of solids-liquid separation, and the chemical aspects which involve chemical modifications to facilitate or improve the separation of solids. The principal unit processes employed for solids removal include:

1. Sedimentation
2. Flotation
3. Filtration
4. Microscreening
5. Coagulation-flocculation
6. Miscellaneous processes which include: moving bed filter, ultrafiltration, magnetic separation, ultrasonic flocculation, etc.

II. Physical Processes

A. Sedimentation

The time-honored method for separation of solids involves sedimentation by gravity. In the conventional horizontal flow sedimentation tank, detention periods of 2 to 4 hours are used to enable suspended particles to settle by gravity. It is the simplest of the processes to remove solids, and it is also the least efficient. Colloidal particles settle at such a slow rate that they are not effectively removed. Some degree of short circuiting always occurs leading to lesser detention times for portions of the flow. Because of the inefficiencies of this process many attempts have been made to improve on the separation, still using gravity as the driving force.

One such improvement is the tube settler developed in this country and the Lamella separator developed in Sweden. Both processes achieve separation by causing the particles to settle only inches rather than the several feet as in the conventional settler. This is accomplished by conducting the wastewater upward thru inclined tubes or plates, the solids move toward the lower end of the tubes while the water passes out of the tops. The tube settler has been rather widely used for separation of floc in chemically treated river water. It is also finding application for removal of solids from chemically treated wastewater. There is sufficient information to indicate that this device does separate particles, but insufficient evidence is at hand to conclude that the increased capital investment over conventional sedimentation alone is warranted.

B. Flotation

Another process which separates particles by gravity is flotation. Separation is achieved by attachment of air bubbles, which effectively reduces the specific gravity of the particles to less than that of water. Flotation has found application for clarification of a number of industrial wastes, however, the process is little used at the present time for clarification of domestic wastewater. Its widest application in wastewater treatment is for sludge thickening operations. With additional development, air flotation may find wider application to raw sewage clarification following flocculation by chemical additives.

Air flotation has some attractive potential advantages over sedimentation: 1) a more positive control over the separation rate by controlling process variables such as air/solids ratio or chemical addition; 2) a lower initial capital cost owing to higher separation rates and shorter detention times; 3) reduction of septicity and associated odors owing to aeration of feed and shorter detention times; 4) greater sludge density allowing use of smaller equipment for dewatering; and 5) multiple use of a single treatment unit for removal of heavy grit, suspended solids and oil or grease. These advantages are gained with the following disadvantages: 1) higher operation costs, and 2) greater operational skill is required. The process clearly needs additional research to define in more detail the above advantages and disadvantages.

C. Filtration

Whenever a high degree of clarification is required, then in-depth filtration after chemical treatment is the process of choice. Rapid sand filtration has been practiced for decades by water treatment plants but only recently for wastewater application. In this process, the wastewater passes through a bed of granular media which captures the particles within the filter. When the capacity to store particles is reached, the filter is restored by backwashing. In an ideal filter for downflow operation, the media is uniformly graded from coarse to fine from top to bottom. The usual sand filter does not meet this ideal requirement, hence mixtures of media have been employed to approach the ideal filter. The most common is a two component filter of coal on top of sand. A tri-media filter contains coal, sand and garnet.

One of the difficulties with filters is that the upper layers of the bed become clogged with solids well before storage capacity is reached in the remainder of the filter. Several approaches have been taken to overcome this problem. The filter can be operated upflow in which case the flow proceeds from coarse to fine media approaching the ideal. Some filters have been designed to introduce the feed into the middle of the filter with flow in two directions.

One of the more promising techniques developed by Johns Manville is described as a moving bed filter. The object here is to renew the sand bed surface either continuously or intermittently to avoid surface plugging. This process has been tested at pilot scale and a full scale installation is being made in Manville, New Jersey. Yet another approach has been proposed by the Research Triangle Institute in which a lightweight media floats to form a packed bed. Wastewater is filtered upflow. As the media becomes clogged, it is removed from the bed, washed, and then reintroduced with the wastewater. The concept is sound but feasibility remains to be tested.

D. Microscreening

Microscreening involves straining of wastewater through a woven metal fabric having openings ranging upwards from 23 microns. The screen is continuously cleaned by pressure sprays. Only larger suspended particles are removed since straining is limited to particle sizes greater than the mesh size. These devices have thus far found their greatest application in treatment of river waters. More recently, application to removal of suspended solids from secondary effluents has been tested. Chicago's Hanover Treatment Plant has successfully operated a microstrainer to reduce suspended solids in secondary effluent to less than 5 mg/l. Since about one-half of the residual BOD of secondary effluent is attributable to the suspended solids content, removal of the solids effects a reduction of the BOD as well as suspended solids.

III. Chemical Processes

A. Metal Coagulants

The colloidal components of wastewater cannot be removed by any of the physical processes described above. To remove these solids, the particles must be coagulated and flocculated to larger size before physical methods can be effective. In conventional secondary treatment the colloids are flocculated by organic polymers produced during the biological oxidation. Coagulation and flocculation can also be achieved by chemical additives.

Chemical coagulation and flocculation were first proposed some thirty years ago but was never widely employed. Today, chemical flocculation is the essential first step in physical-chemical treatment. The use of chemical additives has gained impetus from the need to remove phosphates from wastewater. All metal coagulants now being used for phosphate removal also accomplish clarification.

A wide variety of metal coagulants are suitable for clarification (also phosphate removal). These include: aluminum salts, such as aluminum sulfate, and sodium aluminate; iron salts, such as ferric or ferrous chloride or sulfate, pickling liquor which is an iron-containing waste stream from the steel industry; and lime. Which one of the several coagulants to use in any specific instance cannot be predicted beforehand. All metal coagulants are effective and the choice of one from the many has to be made for any application. The choice for any particular application is generally based on relative dosage, the cost of the coagulant and the chemical composition of the wastewater. It is well to remember that to obtain clarification and phosphate removal in wastewater will require substantial dosages of coagulant which in turn will produce chemical sludges which must find disposal. The range of dosages for iron or aluminum salts range 100 to 300 mg/l while for lime the range is 300 to 600 mg/l or more.

In addition to being the first step in physical-chemical treatment, chemical clarification may have some other benefits in solids removal in the primary prior to biological treatment. This concept is being tested at Grand Rapids, Michigan, at full scale. Some of the advantages that may emerge from this are: decreased air requirement in activated sludge resulting from the increased solids capture in the primary; less difficult-to-filter sludge from the secondary while producing more but filterable solids in the primary. And, of course, phosphates will be removed. One of the advantages of lime is that the sludge can be calcined to recover reuseable lime. This has been demonstrated at Tahoe for lime used in secondary effluent and will be applied to lime sludge from raw sewage precipitation at Rocky River, Ohio.

One of the interesting developments of recent years has been the synthesis of a wide variety of organic polymers. Use of organic polymers or polyelectrolytes as sole coagulants or as aids to the inorganic coagulants has added a new dimension to clarification. Very low dosages of polymer may improve efficiency of solids removal, permit reduction of inorganic coagulant dosages and increase settling rates, thus allowing operation of existing equipment at higher flow rates. Dosages range from fractions of a mg/l to several mg/l. Thus, in contrast to inorganic coagulants, sludge volume is not increased. But organic polymers are not a total panacea. They do not remove phosphates, they are all expensive ranging $1-$2/lb for the 100% product, and their behavior for any particular application is unpredictable. The plant operator is faced with selecting a single polymer from the literally hundreds available and even then he cannot be sure that his choice will be effective all of the time. Polymer clarification of raw sewage has been tried at Cleveland's Easterly Plant and at Grand Rapids.

Whether primarily for clarification or for phosphate removal, chemical addition to wastewater is a growing practice. The resulting chemical sludges will pose problems for their disposal.

IV. Miscellaneous Processes

A number of other processes for solids separation are in varying stages of development. One of these is ultrafiltration which is a process akin to reverse osmosis except that inorganic minerals are not removed. The process involves application of wastewater under pressure to a porous membrane. The process cannot compete economically with other solids removal processes for treatment of large volumes of wastewater. But there are special applications for small volume filtration where ultrafiltration may have application. For example thickening of organic sludges or powdered carbon sludge has been investigated.

Another membrane process, called "cross-flow" filtration by the inventor at Oak Ridge, may be useful for solids separation. In this process a membrane is formed on a support and solids separation is obtained under pressures of 30-50 psi.

V. Assessment for the Future

Research of the last decade has provided the consulting engineer with an arsenal of processes for removal of solids. This development comes at a time when, more than ever, better and cheaper ways of solids removal are required. Phosphate precipitation, improved clarification of raw or secondary effluent, and higher quality effluents for tertiary processes have increased the need for separation processes which are more effective and sophisticated than the simple gravity sedimentation now so widely used.

Of the processes discussed here, media filtration, microstraining and chemical coagulation and flocculation are the processes which are now being used. The other processes will be applied as this technology is improved.

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Sources of Information

Much of the research on processes for removal of suspended and colloidal solids from wastewater is conducted at or out of the Advanced Waste Treatment Research Laboratory. The address of the laboratory and the principal investigators are given below:

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ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

DISSOLVED INORGANIC REMOVAL

PPB 1704

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

DISSOLVED INORGANIC REMOVAL

PPB 1704

PROCESSES FOR REMOVAL OF MINERALS

I. Introduction

During domestic and most industrial uses of water there is added an increment of dissolved inorganic minerals which must be removed if water quality is to be maintained. If recycle of wastewater will be practiced in the future, then almost surely methods will be required to remove inorganic salts. Soluble inorganics are even now a significant problem for many municipalities. For example, a recent survey has shown that of the 20,215 municipal water supplies in the 50 states and 5 provinces in the United States and Canada, 1066 had raw water supplies with a total dissolved solids (TDS) of 1000-3000 mg/l; there were an additional 31 supplies that had a TDS of 300-10,000 mg/l. The rising salinity of many water supplies and the increasing cost of developing alternate sources of better quality make it difficult or uneconomic in many locations to meet the USPHS recommended limit of 500 mg/l TDS for potable water. These factors justify the support for research to develop inorganic removal processes.

Several processes are currently being investigated for reducing the mineral content of municipal wastewater to an acceptable level. These include: (a) ion exchange, (b) reverse osmosis, (c) distillation, (d) electrodialysis, (e) freezing and (f) electrochemical treatment. These processes are in varying stages of development and only the first four mentioned are currently being given serious consideration as practical processes for demineralization.

All demineralization processes produce a brine solution. The disposal of this brine represents a major technical problem in the development of demineralization technology. In coastal areas it may be feasible to discharge brines to the ocean. Solar evaporation in lined lagoons can be employed where climatic conditions are favorable. However, inland areas with limited potential for solar evaporation will require the development of more sophisticated techniques for brine disposal.

II. Ion Exchange

Ion exchangers are materials containing ions that can be replaced by other ions from solution. The replaceable ion carried by the exchanger is known as the counter ion. Carriers of exchangeable cations are called cation exchangers, and carriers of exchangeable anions, anion exchangers. Once all the counter ions are replaced the exchanger is exhausted and must be restored by regeneration with a solution containing the original counter ion.

Ion exchange will almost certainly be an economic process for demineralization of wastewater, if the mineral solids do not exceed 1000-1500 mg/l. This development derives from the commercial availability of new anion resins which have 1) high selectivity for chloride ion, 2) require less regenerant and rinse water, yielding a more favorable ratio of product to feed. But most important has been the discovery that these anion resins do not become "fouled" by organics - the single most important deterrent to ion exchange with the older resins. Up to 50-60% of the COD is removed from secondary effluent with no detectable loss of exchange capacity. The COD is eluted with the regenerant.

Research at AWTRL has confirmed that COD is removed and that fouling does not occur. Studies at the Pomona Pilot Plant facility demonstrated that an effluent containing about 50 mg/l of TDS can be produced from a feed of about 800 mg/l TDS. The bulk of the residual TDS was silica which is not removed by a weak anion resin. Total costs for the process were estimated to be 24¢/1000 gal, excluding the cost for disposal fo the brines. In practice, the product of ion exchange will be blended with good quality, but not demineralized, effluent to provide a product with, say 300-400 mg/l TDS thus yielding a final cost of about one-half of the 24¢/1000 gal cited.

Other cost estimates cited for the DeSal Process (weak anion exchange process developed by Rohm & Haas) have been 18¢/1000 gal for a 1 MGD plant as determined by Rohm and Haas and24¢/1000 gal for a 10 MGD plant estimated by Infilco. Culligan, Inc., is currently investigating several ion exchange processes on a pilot plant scale of 50,000 gpd. Work is also continuing at Pomona and at AWTRL.

The most encouraging work was that done by some Italian workers who came to the following conclusions:

1. The DeSal Process is far superior to conventional ion exchange and makes earlier estimates of cost out of date.

2. Up to 65% of the organic matter is removed and is quantitatively eluted from the resin.

3. After one year's operation no change could be observed on the physical or chemical properties of the resin.

The authors also concluded that ion exchange was applicable to feeds up to 3000 mg/l TDS - a level that we had not generally considered competitive for ion exchange. Of the four methods being considered for demineralization of wastewater, ion exchange will most likely be applied earliest to full scale. The technology is well-developed and the costs appear to be reasonable.

III. Reverse Osmosis

Reverse osmosis is a membrane process in which water is forced to flow from a solution of high salts concentration to one of lower concentration. In natural osmosis, water flows in the opposite direction. Pressures of 600-800 psi are required to obtain this reversal of flow. The earliest applications of reverse osmosis were in the fields of chemical purification and brackish water desalination. The discovery of the cellulose acetate membrane was, perhaps, the single biggest advance in the application of reverse osmosis to desalination.

Membranes are defined as imperfect barriers which "retain" or "reject" molecules of a certain minimum size and will "pass" smaller molecules. The membranes can be tailored to almost any degree of porosity. Several types of materials have been identified as having membrane forming properties suitable for reverse osmosis. Research is continuing on development of more useful membranes.

Cellulose acetate membranes developed for brackish water desalination are relatively tight (i.e. low water permeability) and can reject over 99% of most mineral species. The water flux through these membranes is very low (~ 10 gal/day/ft^2) and are not economic for wastewater demineralization. Moreover, in treating wastewater, the membranes become "fouled" by dissolved and colloidal organic material leading to drastic reduction in flux. These problems have led FWQA to a membrane development program pointed specifically toward wastewater treatment. Most of the effort to date has been in new membrane development and in methods to control flux decline. The most attractive membranes appear to be modified cellulose acetate types. Current judgment is that the optimum membrane will reject 50-75% of the inorganics and 90% of the organics with fluxes of 50-100 gfd. At the same time substantial effort is being directed toward alleviating the fouling problem. Essentially two approaches are being taken: (a) prevention of fouling by pretreatment procedures or by changes in the hydraulics of the system and (b) cleaning methods once the membrane has become fouled. A promising method for the latter is periodic rinsing of the membrane surface with an enzyme solution. Interestingly, the most effective enzyme solutions have been the common commercial detergent pre-soak mixtures such as Biz.

In addition to membranes an extremely important aspect of reverse osmosis is the hardware. Current modules are of several types and configurations: (a) tubular, (b) spiral wound and (c) hollow fiber. Each of these configurations has its advantages as well as disadvantages, and at this point in development no single choice can be made. All are being investigated concurrently. A recent projection of the economics of RO by Kaiser Engineers compared the configuration as follows:

	sq ft membrane / cu ft equipment	flux / gpd/sf	productivity / gpd/cf
Tubular	20	32	640
Spiral wound	250	32	8000
Hollow fiber (nylon)	5400	1	5400
Hollow fiber (CA)	2500	10	25000

From this comparison, it would seem that the follow fiber configurations are superior but in practice hydraulic inadequacies may be a serious drawback.

Another approach to reverse osmosis has been entitled "dynamically formed" membranes. In this development, the membrane is formed either from the constituents of the wastewater or from small additions of a variety of additives. The advantage of these homemade membranes is that they can be destroyed and re-formed whenever the membrane becomes fouled. This work is still in the early stages of development.

Reverse osmosis has enormous potential for wastewater treatment. Theoretically, it is conceivable that most components of wastewater can be removed to a high degree in a single unit process. Typical removals that have been obtained are shown in the following table:

Typical Removals from Secondary Effluent
(CA membrane, 450 psi, ~ 8 gfd)
% rejection

TOC	90	Phosphate	94
TDS	93	Nitrate	65
Turbidity	99+	Ammonia	85
Alkalinity	90	Organic Nitrogen	86
Chloride	80-85		

The practical achievement of the above theoretical capability must await the solution of some serious problems, among which are: membrane fouling, membrane cost, greater (and therefore economic) fluxes, and reduction of operating costs. On the latter, the best estimate is on the order of 40¢/1000 gal projected for brackish water desalination. Because of the potential of this process, research on all of the problems is being pursued vigorously.

IV. Distillation

Distillation is now the most commonly practiced method for obtaining fresh water from sea water. Today there are 90 million gallons per day of plants in operation or under construction in various parts of the world, and this capacity is being expanded rapidly. As everyone knows, distilled water is a common synonym for pure water, hence it is not surprising that distillation is being considered for wastewater treatment and renovation. But distillation of wastewater is substantially different than distillation of sea water. Preliminary studies have revealed that some treatment of the distillate (product) will have to be practiced to remove volatile substances. It is also likely that the solids and organics in wastewater will pose additional problems. All of these aspects are being pursued.

V. Electrodialysis

Another membrane process for demineralization is electrodialysis, but, in contrast to reverse osmosis which uses pressure as the driving force to separate water from minerals, the energy in this case is electrical. A direct electric voltage applied across a cell containing mineralized water will cause the cations to migrate to the negative electrode and the anions to the positive electrode. If cation and anion permeable membranes are inserted between the electrodes, then mineral ions can be separated from the water. Characteristically, 40-50% of the dissolved salts can be removed in a single pass through an electrodialysis stack.

The technical feasibility of electrodialysis has been demonstrated both for brackish water desalination and wastewater demineralization. But, as with reverse osmosis, membrane fouling by wastewater solids and organics has deterred practical application. The process is being investigated at both the Lebanon and Pomona pilot plants of AWTRL. Emphasis of the research is on controlling the membrane fouling by intensive treatment of the feed and by enzyme flushing of the membrane surfaces. The process could be economically attractive once the fouling problems can be solved since cost, exclusive of brine disposal, has been estimated to be 15-20c/1000 gallons.

Assessment for the Future

Almost surely, increasing parts of this country and the world, will look to their wastewater as an additional source of water resources. And just as surely, some form of demineralization will have to be applied to reduce mineral salts. At this time, no single process, of the several being studied, is ready for full-scale application and no single process has a clear and obvious advantage over the others. However, ion-exchange, because of its highly developed technology in other fields, appears to be the process which will find earliest application. A modest breakthrough in reverse osmosis could find this process applied, particularly to certain industrial waste streams. It bears repetition that a suitable method has to be found for disposal of the brine concentrates from any demineralization process.

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Sources of Information

Much of the research on processes for removal of minerals from wastewater is conducted at or out of the Advanced Waste Treatment Research Laboratory. The address of the laboratory and the principal investigators are given below:

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ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

DISSOLVED BIODEGRADABLE ORGANICS REMOVAL FROM WASTEWATER

PPB 1705

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

PURE OXYGEN AERATION OF ACTIVATED SLUDGE

Linde Division of Union Carbide, under contract to FWQA, has completed a comparison of pure oxygen aeration and air aeration in the conventional activated sludge process. The study was carried out in identical parallel trains at the 2.5 mgd Batavia, New York plant. Inefficient utilization of costly pure oxygen has discouraged similar full-scale operation in the past. The covered-staged oxygen injection and dissolution concepts developed by Linde overcome this obstacle and 90-95% utilization of the input oxygen was achieved.

The oxygenation system used employed sealed covers on the aeration tanks and intertank baffles to form a series of staged compartments. Each compartment or stage is equipped with a submerged turbine-rotating sparger unit and a recirculating gas compressor located on the top of the tank cover.

The three points demonstrated by this study with the greatest potential for reducing the cost of waste treatment are:

1. The substantial reduction in aeration volume possible with oxygen aeration while maintaining efficient carbon and solids removal. The oxygen train achieved better treatment in 1-1/2 hours aeration detention time than the air train at 3 hours.
2. The high solid content of the waste activated sludge achieved by the oxygen system; thereby, possibly eliminating the need for a separate thickener operation. Oxygenated sludge had a Sludge Volume Index of 40 and concentrated to about 3% in the final clarifier underflow.
3. The reduced quantity of waste sludge produced with oxygen. Significant reduction in the quantity of waste activated sludge produced by the oxygen system was noted. The best estimates at this time are that the reduction, by weight was 30-40%. Better data on the exact amount will be obtained this summer.

The economic substitution of pure oxygen for air may eventually prove to be one of the most significant breakthroughs in the history of the activated sludge process. The pure oxygen process, in addition to offering potential reduction in new plant construction, is also applicable to many existing high-rate or overload plants which are performing poorly.

For more information, see report "Investigation of the Use of High Purity Oxygen Aeration in the Conventional Activated Sludge Process" by Linde Division of Union Carbide Corporation, Contract No. 14-12-465, or contact:

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TRICKLING FILTERS

There has been no major breakthrough in the past two years. This process is capable of producing a good quality effluent having a BOD_5 of less than 20 mg/l if lightly loaded. In the United States, the tendency is to load the filter at a much higher rate than is done in England. Thus, we find today many installations that will have difficulty in meeting the more stringent water quality standards.

It is not enough to just look for completely new processes, but attention and action must be given immediately to applying known technology to upgrading present treatment plants. All the needed new plants and plant expansion cannot be built in a short time. Substantial amounts of pollution can be prevented from reaching our surface waters by upgrading present plants. There are several ways of achieving higher removals. There is probably no one solution that will work at all installations, for each plant is different. If a plant is not getting good removal and the impairment is not due to toxic or grossly atypical waste, then it is usually due to either hydraulic overload, organic overload, or poor final liquid-solids separation. The following are suggested ways of alleviating these conditions.

Easing hydraulic overload

1. Find and reduce needless sources. Infiltration, downspouts, and cross connection can contribute greatly to the flow.
2. Use large interceptors as holding tanks. Many towns use their main interceptor to the plant to back-up the flow during the day and treat it at night when the flow is low.
3. Construct an equalizing or surge tank to smooth out the high peak flows. An equalization tank will mix and dilute toxic wastes, giving better downstream settling and lessen load fluctuations.

Aiding organic overloaded plants

Most organically overloaded plants can be aided by the same methods suggested for hydraulic overloads since they commonly occur concurrently. Additional methods are:

1. Have industry program the load for slow release. In smaller towns, most industries are willing to program extremely high organic waste flows.

2. Have industry treat at source using a roughing filter or other appropriate means to relieve part of the load.

3. Treat digester supernatant return by alternate methods or program return load to time of low load.

4. Remove more material in the primary tank by using iron or aluminum salts and polymers in the incoming waste. This will also remove phosphorus.

Lessen final solids discharge

One of the greatest improvements that can be made in secondary treatment is reliable solid removal from effluents. For efficient overall removal, the final settler must remove better than 98% of the solids. If overflow weirs are submerged several inches with the present flow, then there is no recourse except to increase settler capacity. For less hopeless cases, the following can be tried.

1. Chemical flocculation or precipitation in process or final effluent treatment.

2. Improve inlet and/or overflow design.

3. Install a microscreener.

4. Install mixed media filters.

5. Install tube settlers.

If a town has a trickling filter that is water tight or can be made so, the filter unit can be simply converted to an aeration tank. This can be done by removing the filter media and installing a surface aerator. The existing primary and final clarifiers can be utilized with minimal structural and piping changes. This type of conversion will usually increase the capacity of the plant twofold for a fraction of the cost of a completely new plant.

All the methods discussed are not new, but are well-proven processes. Thus, there are answers to the question on how a town can meet the new water quality standards. All that is needed is an awareness of the fundamentals involved and a willingness to pay for and use all the technology that is known.

ROTATING BIOLOGICAL DISCS

The rotating biological disc method of treating waste has been used in Europe for at least the last five years. The system basically consists of closely spaced rotating discs alternately submerged in wastewater and exposed to air. Wastewater continuously flows parallel to the discs. The waste level is slightly less than half the disc diameter. The units are usually arranged in series or stages.

The discs are molded of low-density expanded polystyrene. The entire downward load is offset by the buoyancy of the discs. Thus, the only power required to rotate the discs is that needed to overcome bearing friction. Microorganisms attach themselves to the discs and perform the same function as in a trickling filter. The biomass sloughed off the discs is removed in a final clarifier. In short, the rotating biological disc method is a modern version of the "Immersion Filter" developed by Buswell in the middle twenties.

FWQA has funded a grant (1701 EBM) with Rutgers University to assess the degree of treatment and to obtain operating data on this method of treatment. The pilot plant used in this study is a ten-staged unit with a design flow of 8 gpm. This gives a detention period of 5 minutes per stage or a 50-minute overall detention time for the disc unit. The plant has been in operation for about one year at the Jamaica Treatment Plant in New York City near the Kennedy International Airport. Data obtained thus far show that the unit is oxidizing about 93% of the biodegradable carbonaceous matter and 80% of the ammoniacal nitrogen in the primary effluent being treated. A report on the work is not available at this time.

A demonstration grant (11010 EBX) has been awarded to the Village of Pewaukee, Wisconsin to evaluate the effectiveness and efficiency of the rotating biological disc method for treating municipal wastes on a full-scale community level. The performance of the unit will be compared directly with an existing trickling filter under identical conditions. The design flow of the disc unit is 0.46 mgd. The unit is scheduled to be on-stream the latter part of this year.

The rotating disc system has an advantage over a trickling filter unit in that recycle is not necessary at night to keep the biological mass wet because the trough always contains liquid. It seems quite possible that the method can produce an effluent in quality some place between that of a trickling filter and an activated sludge unit. It is conceivable that the system would find application at some of our Federal installations, such as small parks or rest stations where there is a wide variation in the flows. There is a small two-stage unit available that handles population equivalents of 12 to 200 persons.

The main disadvantages of the method are that it must be housed to protect it from storms, hail, etc. and the large disc surface area required. For 90% removal, the unit load is 2.7 gal/day/ft^2 of disc surface area. Normally the discs are ten feet in diameter and the disc spacing is 0.846 inches.

INSTRUMENTATION OF WASTE TREATMENT PLANTS

Instrumentation and control have not yet caught up with the basic requirements of wastewater plants. There are several reasons for the limited use of continuous automatic analysis and control. Some of these are the absence of sensors to measure some of the most important factors directly, the fairly high cost of instruments available, and the willingness of those in the waste treatment field to decide that automatic operation is necessary and to take all the steps required to bring it to fruition. In the past, the cost of instrumentation has eliminated them from consideration by managers of small and medium-sized plants.

Recent emphasis on water quality standards is bringing about a natural increase in the extent of automatic control. This is especially evident in newer facilities where instrumentation is no longer an "afterthought", but an integrated part of plant design. Unfortunately, some engineers engaged in designing new plants have not kept up with the improved processing techniques. The design of a modern plant for treatment of wastewater requires a considerably broader knowledge of treatment and control techniques than in the past.

Many sensors cannot be used in treating wastewater because they become fouled by the gross solids, greases, oil, and aquatic growths. Despite the encumbrances inherent in the physical makeup of raw wastewater and sludge drawoff, measuring devices and instrumentation are now available that can monitor and control most of the secondary plant flow systems. The real problem in automating the various flow regimes is not a lack of flow controlling equipment, but the inability to rapidly measure biological activity or "state-of-health" of the system. For instance, wasting of activated sludge could logically be based on the active mass of microorganisms in the system. However, the closest we can come now to determining active mass is mixed liquor volatile suspended solids and this has been estimated to represent 50 to 100 percent more active solids than are actually present. Thus, the difficulty in controlling the treatment plant is directly attributable to the inability to model constantly changing life processes.

It would appear that the best index for understanding and controlling the activated sludge process would be the amount of living cells in the aeration tank. No method now exists which permits determination of the microbial activity in a manner useful to process control. Adenosinetriphosphate (ATP) is present in and essential to all living cells. Measurement of ATP would be a rapid and unequivocal method for active microbial mass. Biospherics Incorporated is under contract (14-12-149) to design and fabricate an instrument for use in the ATP assay. In addition, they will adapt the firefly bioluminescent method to determine the ATP of activated sludge which is directly proportional to the biomass. E.I. DuPont is now producing commercially the reagents needed for the test; therefore, there will not be any difficulty in obtaining the reagents if the method becomes a reality. This method probably can be automated. The time to perform the tests should be about 15 minutes if done manually.

Biological process efficiency is now measured by various laboratory analytical techniques. The time required to collect, transfer samples, and perform the analyses may take anywhere from three hours to five days. The time involved in obtaining data seriously hinders rapid and effective process control. On-line instrumentation designed to yield reliable, useful information in terms of minutes instead of hours would contribute significantly to improving plant operation. Contracts are now being let to develop an on-line instrument to measure the organic strength of influent and effluent streams at a waste treatment plant. The instrument will be capable of analyzing both filtered and unfiltered samples. This will entail developing an on-line macerating device as well as an on-line filter. Within the next year, it is hopeful that a full automatic on-line COD and TOC analyzer will be available to treatment plants.

A wastewater treatment plant can have too much instrumentation and automation or it cannot have enough. Most wastewater treatment plants now have too little instrumentation to give adequate control. The new pilot plant at AWTRL in Cincinnati will test new process control equipment and instruments in the coming year. The aim here is to operate them under controlled conditions to determine durability, performance, and limitation. This information will then be made available to construction grants people and consultants so that new plants can be operated more efficiently.

ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

MICROORGANISMS REMOVAL FROM WASTEWATER

PPB 1706

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

GPO 821-871-14

Introduction

In this first annual report covering the status of disinfection of wastewater and AWT treatment plant effluents, it is believed that a look at where we stand now and what is planned for the future should provide a better understanding of what may be expected from this subprogram.

Present Status of Chlorination

The most desirable objective is to be able to say that application of a specified dose of chlorine would provide safe disinfection of all effluents. The coliform test should be considered the primary standard; the chlorine residual can only be considered as a secondary standard and it is only valid to the extent confirmed by the results obtained in the coliform test. The conclusions of Browning and McLaren (Jour. Water Poll. Control Fed., August 1967) indicate the problems of operating on a basis of a specified combination of chlorine residual and contact time. They state "Generally speaking, a correlation exists between chlorine residual and coliform density (coliform densities decrease with increased chlorine residuals) but the individualities of waste treatment plants and their effluents make it difficult to apply a correlation determined from one plant to other plants." Each plant must develop its own data for correlating chlorine dosage, residual, and contact time to yield predictably the desired reduction in coliform count.

The most highly clarified and oxidized effluents are the easiest to disinfect. If good control of microorganism content is to be attained by chlorination, good secondary waste treatment should be the minimum. Chlorination of primary effluents should not be considered an acceptable practice in most situations except as an interim process until secondary treatment facilities can be constructed.

Some concern has been expressed regarding the fact that numerous viruses are more resistant to chlorine than the coliform bacteria. Methods of using viruses as an indicator of chlorination efficiency have not reached the stage where practical tests for routine use are available. The coliform test still remains an effective criterion for disinfection of drinking water. Except for hepatitis, clearly defined outbreaks of virus diseases traceable to drinking water have not been reported (Clarke, Berg, et al., Adv. Water Pol. Control Research, Pergammon Press, McMillan Company, New York, Vol. 1, 1964). Epidemics of hepatitis originating in chlorinated water supplies judged satisfactory by the coliform test have not been reported except in instances where obvious deficiencies in chlorination were shown or suspected. It is not, therefore, considered likely that effluents disinfected to satisfactory coliform destruction levels are much of a health hazard. FWQA has funded a grant (69-G385) to investigate the possibility of locating a new bacterial indicator that is sufficiently more resistant than coliform organisms to provide a safety factor for virus destruction. The emphasis is on the discovery of an organism that can be enumerated by simple plate count or MF procedures.

Status of Research

Because of personnel limitations and other problems, research in the disinfection program has been limited in scope thus far. The outlook for the future is improving and a marked increase in the number and variety of grant and contract projects is anticipated in FY 1971.

In-House:

There have been numerous reports in the literature of a major synergistic effect of gamma radiation on the disinfecting action of chlorine, but the work reported has not been adequately controlled. An investigation to determine whether gamma radiation exerts a synergistic effect on the disinfecting action of chlorine is now in progress. This work is being done under very carefully controlled conditions. Present progress indicates that this project will be completed in FY 1971, and it is anticipated that definitive data will be produced to either support or negate the existence of a synergistic effect.

Grants:

Grantee	Subject	Project Director Expected Comp. Date
Illinois State Water Survey, University of Illinois, Urbana, Illinois.	Disinfection of Sewage Effluents with Chlorine and Bromine.	Dr. F. W. Sollo 9/30/70
City of St. Michaels St. Michaels, Maryland. (Clow Waste Treatment Division Aer-o-Flo Yeomans, Melrose Park, Illinois Engineering Operator for Grantee)	Controlled Treatment System-Ultraviolet Disinfection.	John A. Roeber 7/9/70
University of Illinois Urbana, Illinois	New Microbial Indicators of Wastewater Disinfection.	Dr. R. S. Engelbrecht 9/30/71

Much of our research in disinfection of wastewater deals with problems related to the use of chlorine. Chlorine, however, is not necessarily the answer to all of our disinfection problems, and little information is available regarding the use of other disinfectants for the destruction of microorganisms in wastewater. Other disinfectants are, therefore, being investigated. The program is planned to develop, as rapidly as possible, methods for the use of a variety of disinfectants and provide guidelines for their practical application. The rationale for this approach is to make available to the sanitary engineer a spectrum of proven disinfection processes from which he can select the one most applicable to a specific waste treatment disinfection problem.

Research Statements of Need

The extent to which the Disinfection Subprogram can satisfy the needs of the respective Regional Programs depends upon how well we can identify those needs and formulate work programs to satisfy them. Satisfaction of those needs can best be expedited by good liaison with the Region. It would be most helpful if the Regions would submit statements of research needs to cover specific problems in need of solution. The development of an adequate research work plan to satisfy a particular need, however, depends upon the content of the need submitted. This can best be accomplished through a preliminary discussion of the proposed need by the Program Chief and the proponent.

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ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

ULTIMATE DISPOSAL

PPB 1707

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

HE ULTIMATE DISPOSAL RESEARCH PROGRAM

at the

Robert A. Taft Water Research Center
Cincinnati, Ohio

The Ultimate Disposal Research Program is responsible for finding places to put the pollutants which have been extracted from waters by conventional and Advanced Waste Treatments. Disposal of residues must of course be done in ways that will not cause pollution and ideally the residues should be reused to the maximum extent possible. Our responsibilities cover disposal to the air, waters, and to the land, as well as reuse and recycling of constituents. Control of pollution of underground waters is a specific assignment of the Robert S. Kerr Water Research Center, but general considerations of deep-well disposal are also reviewed here. Deep-well disposal is not a generally applicable method and should be used only where the local geological formations are particularly favorable and there are no acceptable alternative methods for disposal.

Disposal of wastes is the most frequently neglected part of our modern industrial civilization, and is directly responsible for our polluted planet. Attention to unit operations, with little regard for the fate of by-products which are no longer interesting and are frequently embarassing, has produced the present situation. In an attempt to focus attention on disposal problems, and to reduce some forms of pollution at their sources, a number of papers have been published calling attention to valid and invalid disposal methods (see Bibliography).

Disposal of organic sludge from conventional wastewater treatment plants accounts for up to 50% of the total costs of treatment. Disposal of sludge requires removal of the water content which accounts for 95 to 99.5% of the weight, followed by storage of oxidation of the organic matter. Since activated sludge has a much lower solid content than primary sludge, the addition of secondary treatment greatly increases the sludge disposal problems of the plant. Equipment which was effective for primary sludge frequently proves to be inadequate when waste activated sludge is added. The current FWQA policy, requiring secondary treatment for most large plants discharging to inland waters, will greatly magnify the sludge disposal problems in this country.

Incineration can be accomplished in modern equipment without producing pollution of the air or water. An outstanding example of pollution-free incineration may be seen at the South Lake Tahoe Advanced Waste Treatment Plant (see "Product Recovery" in attached list of contracts). At Lake Tahoe, organic sludge and lime sludge are separately incinerated in two incinerators which produce absolutely no plume or odor. It is impossible to tell from outside the plant whether the incinerators are working or not.

The cost of incinerating sludge is directly dependent on the water content; therefore, efficient dewatering is the key to efficient incineration. Present dewatering techniques include sedimentation, vacuum filtration, pressure filtration, and centrifuging. All of these techniques are being examined in full-scale equipment under demonstration grants. There are a number of chemicals which aid dewatering, including lime and salts of iron and aluminum, as well as synthetic polymeric flocculants. A limited amount of laboratory work is devoted to evaluation of available products, but it is recognized that field experience provides the most usefuly information. Radiation, freezing, pressure cooking, treatment with enzymes and the addition of sludge ash have all been proposed as useful aids to dewatering. Both radiation and freezing have turned out to be too expensive for the benefits achieved. Pressure cooking with or without the addition of oxygen is being evaluated at Colorado Springs, Painesville, Ohio and Santee California. Pressure filtration with ash is being evaluated at Cedar Rapids, Iowa, and vacuum filtration has been studied in the laboratory. A contract for the development of enzymes to thicken sludge has been underway at Aerojet-General.

Land disposal of sludge without previous dewatering is particularly attractive, since it uses a low cost filter, the earth, and low temperature oxidation by microorganisms. Although larger quantities have to be transported when the sludge is not first dewatered, the transportation can be done economically by pipeline. A recent contract with Bechtel, Inc. appraises the cost of pipeline transport of sludges for the case of Cleveland, Ohio, but can be applied to many other situations. Land disposal of sludge if properly operated is true conservation of resources and recycles essential elements to the biosphere. On low grade land, sludge improves fertility and enhances the value of the environment. Political objection to other people's "sludge" and a natural aversion to old fashioned smelly sewage farms has held back rational utilization of the land for sludge disposal even though its cost may be as low as one fourth of that for drying and incineration. Papers have been published and talks given to point out the advantages of land disposal. Greenhouse and field plot studies are underway, both in-house and under grants, to improve our knowledge of land disposal. A recent workshop at Chicago reviewed the state of the art of land disposal and an excellent summary has appeared in the May 16, 1970 issue of the Prairie Farmer.

There are valid objections to land disposal of sludges, particularly if they are improperly applied. Excessive loadings of sludge can contaminate ground water and in some situations it may be necessary to collect and treat water from underdrains as it is done in major irrigation projects. Nitrates are apt to be the principal pollutant, just as they are with irrigation. There is little evidence that the soil will be poisoned by excessive quantities of organic matter or by heavy metals in the sludge, if proper care is taken. Pathogens can be controlled by pasteurization if necessary, or by long holding periods in lagoons. Even without these protective measures there have been no reports of sludge born disease since 1919, despite widespread application of sludge to farm lands in this country and in Europe. Studies of pathogen survival in soil are underway.

Sludge can not be stored without some form of stabilization to prevent putrefaction and the development of objectionable odors. Anaerobic digestion is a reasonably well understood process that causes a great deal of difficulty, particularly when practiced on a small scale. Aerobic stabilization is potentially capable of destroying nitrogen compounds and appears to be an ideal pretreatment for land disposal. The process is poorly understood, and the costs of aeration are high. Some in-house work on aerobic stabilization has been carried out and we are looking for a suitable contractor to investigate this process further. Treatment with lime is an attractive alternative to digestion preceding land disposal.

Petroleum wastes represent an interesting application of land disposal. Despite common knowledge that oil "kills the soil", well-oxidized soil will destroy up to 12 inches of petroleum by-products in a year under favorable conditions of temperature and humidity, provided that the soil is kept well-aerated (Dotson, et al., 1970).

Industrial sludges frequently consist of soil minerals such as calcium, carbonate and sulfate and oxides of iron and aluminum. There minerals can be incorporated into the soil in substantial quantities without destroying the agricultural value of the land. In all applications of wastes to the land it is necessary to have good farm management, and not to treat the land as a dump.

The land treatment of sewage is of interest for small communities. This process is frequently referred to as the "living filter". Extensive work on this application has been done at Pennsylvania State University and by several food processes. We have recently received a number of proposals for grants to use the "living filter" treatment for phosphate removal in the Great Lakes area.

Disposal to the atmosphere should be limited to gases which are naturally present. In addition to nitrogen, oxygen, water vapor, and carbon dioxide these include ammonia, nitrous oxide (N_2O), and products of combustion such as higher oxides of nitrogen and sulfur. The last two groups are well-recognized air pollutants, but are contributed in insignificant quantities by well-operated incinerators. Nitrous oxide is remarkably inert and is probably not an air pollutant. Ammonia is rapidly absorbed by moisture and vegetation and reacts with the oxides of sulfur, preventing formation of sulfuric acid. It could be an objectionable nutrient if released upwind of a large body of water, but it could reduce air pollution in some industrial areas. We are studying the removal and destruction of ammonia under a contract with Battelle-Northwest.

Conversion of nitrates to nitrogen gas by dilute solution reduction has been the subject of another contract which is being brought to a close. The process does not appear to be practical for municipal effluents but may be useful for certain industrial wastes.

Brine disposal can contribute a significant part of the cost of water renovation in inland areas. The problem of brine disposal at three major western cities is being evaluated under a contract with Burns and Roe. If salt water lakes or playas are not available, it may be necessary to evaporate the bulk of the water and transport the remaining slurry to a salt-water area. Evaporation ponds appear to be the best solution in arid areas where desalting is most necessary. A small evaporation pond study is being carried out as a part of our cooperative research program with LACSD at Pomona. We are currently regotiating a contract to evaluate the potential of cooling towers for evaporating bring solutions, particularly in high rainfall areas.

Recovery and reuse of the chemical values in sludge is esthetically attractive, but only in rare cases are sludges attractive sources of raw materials for other operations. Utilization of sludge to increase the agricultural value of land is, of course, one form of reuse which is economically attractive. At the other end of the scale, although sludge is a good source of Vitamin B-12, extraction of this substance does not significantly reduce the problem of sludge disposal. The sale of dried sludge for agricultural purposes at Chicago defrays only a quarter of the cost of drying sludge. Recovery of chemicals used to treat sewage or sludge is seldom able to pay the cost of recovery. Lime recovery at Lake Tahoe is probably no better than a break-even proposition; however, it reduces the sludge disposal problem significantly. We are investigating under contract possible markets for the phosphate-rich fraction of the recovered lime which would otherwise be disposed of in a land-fill. In-house studies of aluminum recovery from sludges containing aluminum that was used to remove phosphates indicate that in most situations the recovered aluminum salt would cost more than new chemical; however, the recovery process greatly improves the dewatering qualities of the sludge and may be economical from that point of view. We have also found that recovered lime is superior to fresh lime on a calcium hydroxide basis when used for phosphate removal. The recovered lime produces sludges which are easier to filter and centrifuge because of the presence of inert filter aids formed by the ash.

A small contract has studied the recovery of amino acid values from sludge and their utilization as animal feed. Activated sludge is essentially a form of single-cell protein containing, unfortunately, substantial quantities of undigestible matter. If the nutritive amino acids and sugars can be economically separated from the undigestible fraction, it should be possible to make a valuable feed supplement (Dean and Bouthilet, 1970).

Attached is a list of references to pertinent publications from the Ultimate Disposal Program and a list of contracts dealing with significant aspects of our work.

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TWRC-5 (WPRD 26-01), March 1969. "Ammonia Removal from Agricultural Runoff and Secondary Effluents by Selected Ion Exchange", Battelle-Northwest.

TWRC-8 (14-12-413), March 1969. "Evaluation of Operating Parameters of Alumina Columns for the Selective Removal of Phosphorus from Wastewaters and the Ultimate Disposal of Phosphorus as Calcium Phosphate", Battelle-Northwest.

TWRC-10 (14-12-415), September, 1969. "Mathematical Model of Sewage Sludge Fluidized Bed Incinerator Capacities and Costs", General American Transportation Corporation.

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ADVANCED WASTE TREATMENT RESEARCH LABORATORY
CINCINNATI, OHIO

CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

WASTEWATER RENOVATION AND REUSE

PPB 1708

Division of Process Research & Development
Federal Water Quality Administration
U. S. Department of the Interior

1970 STATUS OF WASTEWATER RENOVATION AND REUSE

PPB - 1708

Reuse applications of wastewater include irrigation, formation of recreational lakes, industrial uses, groundwater recharge for a variety of reuses, and direct domestic reuse. Some of the applications can now be considered well established. Others are only beginning to be considered. There is, however, some activity in each area at this time.

IRRIGATION

Use of biologically treated wastewater for irrigation of non-edible crops and for parks and golf courses has become fairly widespread. With the technical feasibility of this application no longer in doubt, it should become more common in the future. Use of wastewater has the benefit of supplying significant amounts of plant nutrients, thus reducing fertilizer requirements. R&D Grant projects at Colorado Springs, Antelope Valley near Los Angeles, Irvine Ranch, California, and South Lake Tahoe, California include production of water for irrigation. An area in which wastewater is not used is for irrigation of food crops. Studies are needed to define better the water quality for this application.

RECREATIONAL LAKES

Filling of recreational lakes with renovated wastewater was begun at Santee, California in 1961. The success of that project has resulted in the establishment of several other lakes and many more are now being planned. To use wastewater for this application requires at least biological treatment and phosphorus removal. At Santee, phosphorus removal was first accomplished by passing biologically treated water through natural gravel beds. This method will probably be replaced by chemical precipitation using lime. Work at the site is being supported by an R&D Grant.

At Antelope Valley, California filling of a recreational lake with renovated wastewater was begun early this year. Treatment of the water at this location includes oxidation ponds and chemical clarification with alum. Development of the treatment system was partly supported by FWQA. The full scale project is being supported by an R&D Grant.

Another recreational lake project is that at South Tahoe. Indian Creek reservoir receives very high quality water from the Tahoe advanced waste treatment plant. The water is secondary effluent that has been clarified using lime and carbon treated.

The use of wastewater for recreational lakes can often be combined with irrigation. The lake merely serves as a reservoir for the irrigation water. The Tahoe site is an example of this dual purpose reuse.

INDUSTRIAL REUSE

Industrial reuse of wastewater represents a very large potential application. The largest single industrial use of water is for cooling water with an estimated annual volume of 57,000 billion gal. Two other important uses are for process water and boiler water feed.

The only major industrial reuse of wastewater up to this time has been for cooling water. For a number of years the Bethlehem Steel Company plant near Baltimore has been using secondary effluent for this purpose. Generally, it has been found that phosphorus removal is required for the water to be acceptable. At Baltimore the peculiar composition of the water allows phosphorus removal to occur during biological treatment. At other locations, such as at Las Vegas where the Nevada Power Company uses wastewater in their condensers, tertiary phosphorus removal is required. Lime treatment is used at the latter site.

FWQA did not support the early work on reusing wastewater for cooling purposes. Presently, however, Colorado Springs is receiving R&D Grant support for work in this area. A recently funded R&D Grant to Contra Costa County, California also includes reclamation of wastewater for cooling.

There is a need to investigate the use of wastewater for other industrial applications. The recent R&D Grant to Contra Costa County includes study of wastewater for boiler water feed. Work at that location may be extended to other industrial uses. More projects of this nature appear justified.

GROUNDWATER RECHARGE

An increasingly serious problem in water short areas is the lowering of the groundwater level. This occurs because water is pumped out but is not replaced. In coastal areas the result can be intrusion of seawater into aquifers making them unusable. In other locations brackish water may eventually replace the water removed.

It has been recognized that renovation of wastewater and recharge of this water may be a practical method for overcoming the problem. Recharge may be carried out by surface spreading of the water or injection into a well. In the Los Angeles area, surface spreading is being practiced. Recharge was begun in 1962 of the effluent from the Whittier Narrows Plant. This plant, operated by the Los Angeles County Sanitation Districts, produces a very high quality secondary effluent. Because of the porous nature of the spreading surface, no further treatment has been found necessary. Additional biological oxidation and nitrification of the effluent do take place during percolation through the soil. The quality of the renovated water is further improved by dilution with the natural groundwater.

In locations where the percolation rate is low or where spreading areas are not available, well injection would be necessary. Care must be taken in these cases to assure that the water is of proper quality to be compatible with the strata of the aquifer, i.e., will not form precipitates which clog the area around the well. Furthermore, the water must not contain suspended matter that will cause clogging. Orange County, California has experimented with injection of wastewater to decrease seawater intrusion. Treatment of the wastewater consisted of oxidation in a trickling filter followed by alum clarification. Nassau County, Long Island is studying injection for prevention of seawater intrusion and for other uses. This work is being supported by an R&D Grant. Treatment of the wastewater at this location consists of activated sludge, alum clarification, and granular carbon treatment. Nitrogen removal is also being considered.

DOMESTIC REUSE

Reuse of wastewater for domestic purposes involves both non-potable and potable applications. Non-potable use is not new and is no longer rare. Since 1925 treated wastewater has been used for flushing and other purposes at the Grand Canyon. Treatment consists of activated sludge, coal filtration, and chlorination. Similar systems are being used in other water-short resort areas. A biological treatment unit followed by membrane filtration is being tested at Pikes Peak. This work is being supported by an R&D Grant. It produces water of high clarity.

Instances of indirect potable use of renovated wastewater, such as occurs when a municipality practices water recharge, are increasing. In these cases there is usually a large amount of dilution water. The situation is similar to that occurring in many cities where river water containing effluents from cities upstream is used for the water supply.

The concept of direct reuse of wastewater for potable water has been discussed at length by many authorities in the water field for a decade or more. Essentially no direct reuse was actually carried out, however, until 1969 when a renovation plant at Windhoek, Southwest Africa began operation. For more than a year this plant has been supplying about one-third of the total water supply. The treatment system includes biological oxidation by trickling filter, further oxidation in maturation ponds, algae separation by alum flotation, foam fractionation for removal of foaming contaminants, filtration, carbon treatment for removal of remaining organic materials, and breakpoint chlorination for removal of any residual ammonia and for disinfection. This pioneering operation will have an important bearing on the growth of direct wastewater reuse. Other African communities are very much interested in similar projects. Continued success at this location should contribute significantly to the acceptance of this reuse concept.

OTHER SOURCES OF INFORMATION

Many articles have been written recently about reuse of wastewater. While these have dealt at length with the philosophy of reuse and possible treatment systems, they have not often reported actual reuse results. Much practical information on reuse has, undoubtedly, never been published. This information is of great importance to municipalities and to potential reuse customers in making decisions. There is a strong need to collect and analyze existing reuse results and to make them available in unified form.

A number of reuse articles have been collected in "Water Reuse", Chemical Engineering Progress Symposium Series No. 78, Vol. 63, 1967. Reading of this publication is recommended.

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ADVANCED WASTE TREATMENT RESEARCH LABORATORY
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CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

WASTE TREATMENT OPTIMIZATION

PPB 1709

Division of Process Research & Development

Federal Water Quality Administration

U. S. Department of the Interior

GPO 821-571-15

Waste Treatment Optimization
PPBS Category 1709

The four principal areas of activity for PPBS Category 1709, Waste Treatment Optimization are shown in Figure 1. A list of in-house reports completed is shown in Table I.

Design and performance technology is principally concerned with finding quantitative expressions for performance and cost of wastewater treatment processes as a function of the nature of the wastewater to be treated and the decision variables associated with the individual processes. These quantitative relationships take the form of mass balance relationships for all of the elementary chemical and physical constituents of contaminant present in the water, rate of reaction equations, and equations expressing the separation efficiency between liquid, particulate, and gaseous phases. Normally, a group of equations is required to express the performance of the process operating over the full gamut of operating modes and design decisions. This group of equations is often referred to as a mathematical model for the process. Mathematical models can be steady-state, quasi-steady-state, or time-dependent. Time-dependent models are of interest when the quality of the effluent stream from the process as a function of time is important or when the effectiveness of various kinds of control schemes is being considered. The computational procedure for solving all of the quantitative equations simultaneously is usually too laborious to be accomplished by hand calculation. The digital computer is, therefore, used in most cases. Expressing the models as computer programs has the additional advantage of packaging the information in succinct form readily usable by design engineers and planners.

A list of reports produced as a result of in-house activity is shown in Table I. A list of reports which have been completed as a result of contracting activity is shown in Table II. Only three of these contractor reports are now available for distribution. Other contracts in force will produce models for multiple hearth incineration of sewage sludges and microscreening. Contracts in force will also produce capital and operating and maintenance cost data for all of the conventional processes as well as a cost estimating guidelines manual and a staffing guidelines manual.

TREATMENT OPTIMIZATION RESEARCH PROGRAM

I. DESIGN AND PERFORMANCE PREDICTION TECHNOLOGY

1. Develop quasi-steady-state and time-dependent models for preliminary design and simulation
2. Validate design and simulation models by comparison with detailed measurements on operating plants
3. Develop quasi-steady-state models into a recognized standard of performance for use by governmental agencies for regulation and administration of grant-in-aid programs

II. OPERATION, MAINTENANCE, AND PLANT MANAGEMENT TECHNOLOGY

1. Plant performance standards and effluent quality control methods
2. Plant management, training, and staffing criteria and methods
3. State, County, or Regional systems for management and regulation

III. AUTOMATIC CONTROL FOR PLANTS

1. Study feasibility of proposed control loops with time-dependent models to solve transient problems
2. Study cost-effectiveness trade-off between automation and additional or better trained staff or better managerial surveillance
3. Demonstrate and evaluate control schemes on a loop-by-loop basis
4. Demonstrate interprocess control of complete plants

IV. COST-EFFECTIVENESS STUDIES

1. Selection of processes and design policies for least cost
2. Collection and organization of basic cost information
3. Develop recommended cost guidelines for cost estimation

FIGURE 1

TABLE I

PRINCIPAL REPORTS PRODUCED BY TREATMENT OPTIMIZATION RESEARCH PROGRAM

1. Smith, Robert, "Preliminary Design and Simulation of Conventional Wastewater Renovation Systems Using the Digital Computer", FWPCA Publication No. WP-20-9 (March, 1968).
2. Smith, Robert, "Cost of Conventional and Advanced Treatment of Wastewater", FWPCA Publication (July, 1968).
3. Smith, Robert, Eilers, Richard G. and Hall, Ella D., "Executive Digital Computer Program for Preliminary Design of Wastewater Treatment Systems", FWPCA Publication No. WP-20-14 (August, 1968).
4. Roesler, Joseph F. and Smith, Robert, "A Mathematical Model for a Trickling Filter", FWPCA Publication No. W69-2 (February, 1969).
5. Smith, Robert and McMichael, Walter F., "Cost and Performance Estimates for Tertiary Wastewater Treating Processes", FWPCA Publication (June, 1969), TWRC-9 Released January 15, 1970.
6. Roesler, Joseph F., "Preliminary Design of Surface Filtration Units (Microscreening)", FWPCA Publication (June, 1969).
7. Smith, Robert and Eilers, Richard G., "A Generalized Computer Model for Steady-State Performance of the Activated Sludge Process", FWPCA Publication (October, 1969).
8. Smith, Robert, "Factors to be Considered in Developing a Data Gathering and Analysis Plan Leading to Improvement of the Operational Effectiveness of Conventional Wastewater Treatment Plants", FWPCA Publication (December, 1969).
9. Roesler, J. F., Smith, R. and Eilers, R. G., "Mathematical Simulation of Ammonia Stripping Towers for Wastewater Treatment", In-House Report.
10. Smith, Robert and Eilers, Richard G., "Simulation of the Time-Dependent Performance of the Activated Sludge Process Using the Digital Computer", In-House Report 90% Complete.
11. Smith, Robert and Eilers, Richard G., "Cost to the Consumer of Collecting and Treating Wastewater in the United States", In-House Report (July, 1970).

TABLE II

1. "Cost of Wastewater Treatment Processes", TWRC-6, Dorr-Oliver, Inc.

2. "Mathematical Model of Tertiary Treatment by Lime Addition", TWRC-14, General American Research Division/General American Transportation Corp.

3. "Mathematical Model of Sewage Sludge Fluidized Bed Incinerator Capacities and Costs", TWRC-10, General American Research Division/General American Transportation Corp.

4. "Mathematical Model of the Electrodialysis Process", Process Research, Inc.

5. "A Mathematical Model of a Final Clarifier for the Activated Sludge Process", Rex Chainbelt Inc.

6. "Ammonia Stripping Mathematical Model for Wastewater Treatment", IIT Research Institute.

7. "Mathematical Model of Recalcination of Lime Sludge with Fluidized Bed Reactors", General American Research Division/General American Transportation Corp.

8. "Mathematical Model for Wastewater Treatment by Ion Exchange", IIT Research Institute.

9. "Methodology for Economic Evaluation of Municipal Water Supply/Wastewater Disposal Including Considerations of Seawater Distillation and Wastewater Renovation", Bechtel Corp.

10. "Mathematical Model for the Reverse Osmosis Process", Aerojet-General Corp.

As a result of the work reported in the first report in Table I, it was realized that a tool was needed which would allow the process designer to select the group of processes and the piping arrangement to be used and then calculate the performance and cost of the system as a whole. To meet this need an Executive Program was developed as described in the third report of Table I.

By iterative techniques the Executive Program calls each process subroutine in turn and recomputes all recycle streams until the correct solution for the system is found. Performance and cost for each process and for the system as a whole is printed. This program is simple in concept and requires a digital computer with a core memory of about 16K words.

Every quasi-steady-state model developed will ultimately be included in the Executive Program. A list of the individual processes to be included in the Executive Program are shown in Table III with the status of each model. No advanced processes are included in the Executive Program at the present time although several subroutines have been developed.

Preferred advanced or tertiary wastewater treatment systems are shown in the fifth report of Table I (TWRC-9). Estimated removal efficiency for all significant contaminants are given together with capital and operating and maintenance cost.

A generalized model for the activated sludge process has been developed and has been shown to fit data from a wide range of process modifications from the short detention time, low mixed liquor suspended solids, "modified process" to the "extended aeration process". This model is described in the seventh report listed in Table I. The most significant discovery associated with this work was that the maximum rate constant for synthesis is not a true constant but varies significantly with the loading on the process.

A time-dependent model for the activated sludge process has also been completed and the report on this model is about 90% complete. Three classes of active solids are considered; heterotrophs which convert biodegradable carbon to new cells, Nitrosomonas which converts ammonia nitrogen to new cells and nitrite, and Nitrobacter which converts nitrite to new cells and nitrate. This model has been used to investigate a number of schemes for automatic control of the activated sludge process. The most practical of the schemes involve sludge storage in the stabilization tank.

Another model for which a report has recently been completed is the model for ammonia stripping and cooling towers. The ammonia stripping portion of the program is embedded in the cooling tower calculation in order that the variation of Henry's Law constant with water temperature can be taken into account. The program can be used to calculate either ammonia stripping tower performance or cooling tower performance. Both crosscurrent and countercurrent towers are simulated. A numerical integration technique is used in which the tower is divided into cubical elements. Performance from various sources is being used to find the height of a transfer unit as a function of the type of packing and design decisions. Experimental data received from the Marley Co. for a particular packing have been analyzed to find the relationship between height of a transfer unit and the liquid and gas loading, (lb/hr/sq ft). The height of a transfer unit was found to depend on the ratio of gas to liquid loading as follows:

$$\text{Height of Transfer Unit, ft} = 4.1272\ (\text{Gas loading/liquid loading})^{1.257}$$

Various in-house and contract activities are underway to develop operation, maintenance, and plant management policies and methods which can be used to assure that a level of performance commensurate with the capability of the installed treatment works will be consistently achieved. The eighth report listed in Table I deals with these problems. Various contracts are either funded or being considered for funding.

The State of Minnesota has shown an interest in developing and demonstrating a computerized system for surveillance and regulation of treatment works within the State. The system would make use of all design and simulation relationships known to be valid for treatment processes. The physical characteristics of each particular treatment plant would be stored in the computer program. Design and simulation relationships would be used to compute the expected performance of each plant as a function of the measured influent stream. The transient nature of the feed stream and the stochastic aspects of performance relationships would be used to compute a range of expected performance. Monthly performance reports submitted by individual plants would be analyzed and evaluated in a matter of minutes. If deficiencies are detected some sort of remedial action could then be initiated.

Our approach to automatic control of plants is to study the cost and effectiveness of each individual control loop. Performance must be measured and documented with and without the control loop installed. Time-dependent mathematical models will be used to study the feasibility of untried control loops and to study the significance of time-dependent measurements made on individual processes. The tenth report in Table I deals with a time-dependent model for the activated sludge process.

Various cost-effectiveness studies are undertaken to show the cost contribution of various process system components and to study the cost-effectiveness trade-offs for competing process systems. The influence of size of community and the contribution of ancillary elements such as Customer Services and Accounting or General and Administrative Expense are studied to show the general cost perspective. A recent report shown as number eleven in Table I deals with these ancillary costs. Three selected figures from this seventy page report are shown in Figures 2, 3, and 4.

TABLE III

PROCESS SUBROUTINES TO BE INCLUDED IN THE EXECUTIVE PROGRAM

CONVENTIONAL PROCESSES

I. Physical Processes

1. Conveyance for Ultimate Disposal
   a. Pipelines
   b. Truck and Rail Transportation RFP (1/15/70)
   c. Ocean Outfalls

2. Sewage Pumping Facilities (In-house Task)

3. Pretreatment
   a. Bar Screens
   b. Comminution (In-house Task)
   c. Grit Removal

4. Primary Sedimentation (Completed)

5. Sludge Drying Beds (In-house Task)

6. Post and Pre Aeration (In-house Task)

II. Biological Processes

1. Activated Sludge Process (Completed)

2. Trickling Filter Process (Completed)

3. Waste Stabilization Ponds
   a. Aerated Lagoons
   b. Facultative Ponds RFP (1/15/70) + (In-house Task)
   c. Oxidation Ditches

4. Anaerobic Digestion (Completed)

5. Aerobic Digestion RFP (1/15/70)

III. Physical-Chemical Processes

Gravity
1. Thickening of Organic Sludges RFP (1/15/70)

2. Centrifugation of Organic Sludges (Contract 515 Underway)

3. Flotation Thickening of Organic Sludges (Proposal Recommended)

4. Vacuum Filtration of Organic Sludges RFP (1/15/70)

5. Use of Chemicals to Promote Sedimentation (No Plans)

6. Elutriation of Organic Sludge (No Plans)

7. Multiple Hearth Incineration of Sludges (Contract 547 Underway)

8. Fluidized Bed Incineration of Organic Sludge (Completed)

9. Wet Oxidation of Organic Sludge (No Plans)

ADVANCED PROCESSES

I. Physical Processes

1. Cooling Towers (Completed)

2. Microscreening (Contract 819 Underway)

3. Rough Filtration of Secondary Effluent RFP (1/15/70)

4. Dual Media Filtration RFP (1/15/70)

II. Biological

1. Disinfection
    a. Chlorine
    b. Iodine (No Plans)
    c. Ozone

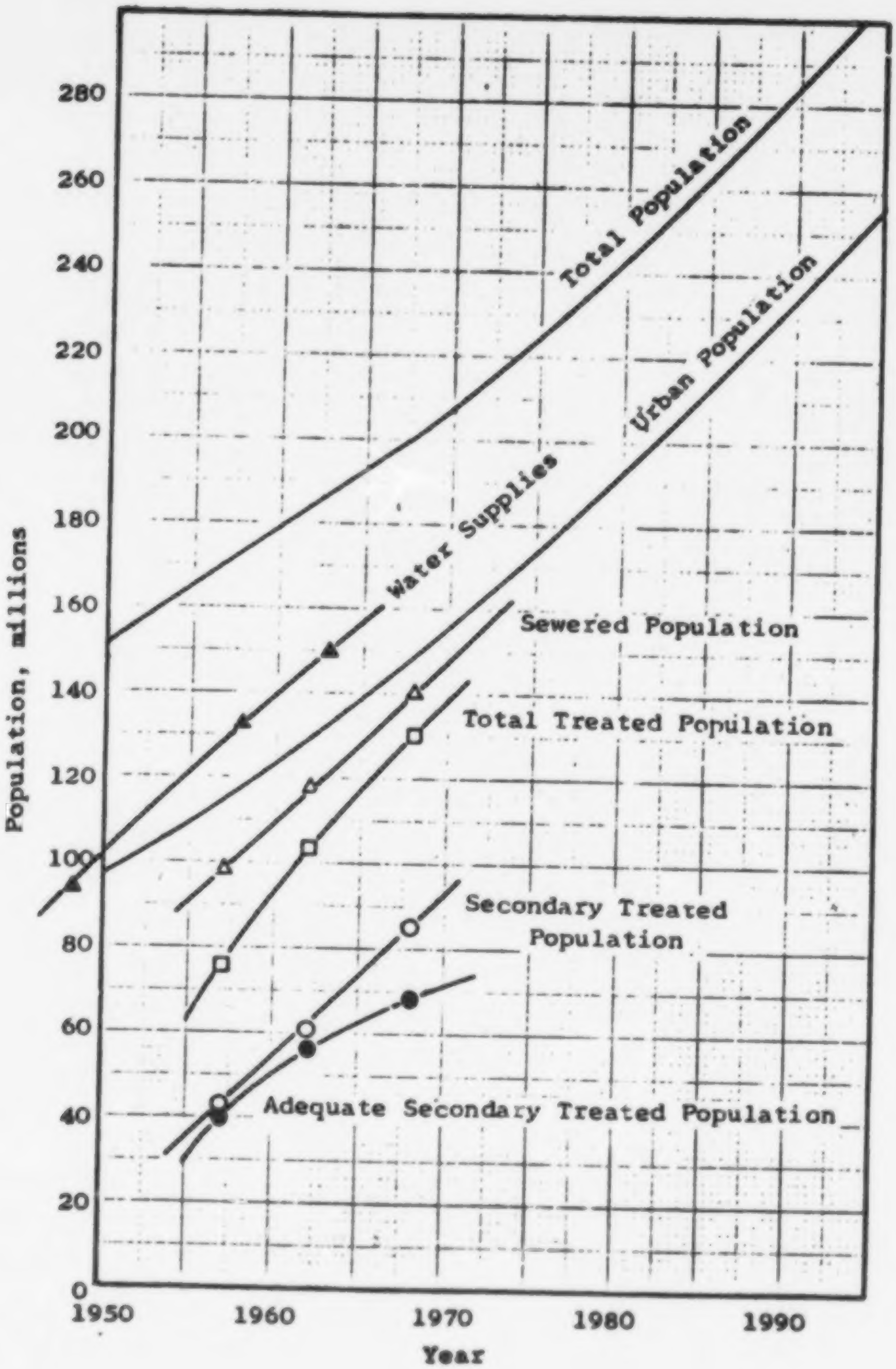
2. Denitrification in Columns RFP (1/15/70)

III. Physical-Chemical Processes

1. Lime Clarification (Completed)
    a. Recalcination of Lime Sludge (Fluidized Bed Complete)
    b. Recarbonation using CO_2 (In-house Task)

2. Ammonia Stripping Towers
    a. Countercurrent (Completed)
    b. Crosscurrent (Completed)
    c. Aeration (In-house Task)
    d. Biological Activity (In-house Task)
    e. Scaling (In-house Task)

3. Granular Carbon Adsorption RFP (1/15/70)
4. Powdered Carbon Adsorption (No Plans)
5. Electrodialysis (Completed)
6. Reverse Osmosis (Completed)
7. Ion Exchange (Model Complete - In-house Task Req'd)



STATUS OF MUNICIPAL WASTEWATER TREATMENT FACILITIES
IN THE UNITED STATES

FIGURE 2

Type of Treatment

	Activated Sludge	Interceptors and Outfalls	Trickling Filter	Primary Sedimentation	Upgrading From Primary to Activated Sludge	Stabilization Ponds
Total Sewered Population	28.95	29.88	29.46	17.71	17.49	5.23
Activated Sludge and Extended Aeration	25.53	29.49				
Trickling Filter			45.14			
Primary Sedimentation				16.04	15.10	
Stabilization Ponds						21.42

NATIONWIDE AVERAGE CONSTRUCTION COST, DOLLARS PER CAPITA (1968 DOLLARS)

Source: cost data - R. L. Michel, Construction Grants and Engineering Branch, FWQA
population distributions - 1968 Inventory of Municipal Waste Facilities in the U. S.

FIGURE 3

FIGURE 4

TOTAL COST OF SEWAGE COLLECTION AND TREATMENT IN 1968 ON A CONTINUOUS CASH FLOW BASIS

1968 dollars/capita/year

Amortization Cost		
House Connection	$ 1.38	
Municipal Sewers	$ 8.64	
Interceptors and Outfalls	$ 2.46	
Treatment Plants	$ 2.83	
Total Amortization Cost	$15.31	
Current Expenses		
Municipal Sewer Maintenance		$0.86
Treatment Operation and Maintenance		$1.55
Customer Service and Accounting		$0.71
General and Administrative		$1.37
Total Current Expenses		$4.49
Total Cost of Municipal Collection and Treatment	$19.80	
Imputed Cost of Industrial Wastewater Treatment	$ 5.05	
Total	$24.85	

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ADVANCED WASTE TREATMENT RESEARCH LABORATORY
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CURRENT STATUS OF ADVANCED WASTE TREATMENT PROCESSES

July 1, 1970

SCIENTIFIC BASES OF WASTE TREATMENT PROCESSES

PPB 1700

Division of Process Research & Development
Federal Water Quality Administration
U.S. Department of the Interior

Status of Research on
Scientific Bases of Waste Treatment Processes

Waste treatment is a chemical process industry. Its function is to treat a starting material (sewage) of some chemical composition by optimum processes to convert it to another material of higher economic value/lower nuisance effect (treated effluent). Optimum processing requires adequate knowledge of the chemistry, physics, and biology of the raw material, treatment agents, and final product.

The major research effort has been on composition of wastes and its changes. Though sewage has been analyzed for nearly a century, our background knowledge is slight and expressed in quite general terms. An intensive analytical program has begun only now, both in-house and by contracts and grants. A first effort has been to determine how to sample effluents and transport samples to the laboratory. Freeze-concentration, though highly praised, is unsuitable. Vacuum concentration is the only practical means available so far. Though a systematic analytical program is evolving, we have leapfrogged to some more specific approaches. One of these, just begun by contract, consists of liquid chromatography of primary and secondary sewages, yielding fingerprint chromatograms. First trials show some 50 - 75 separated organic components, with conspicuous differences developing during biological treatment. Another of these leaps involves developing specific analyses for contaminants of special significance in sewage. Methods were developed for residual polymeric coagulant in treated sewage and for nitrilotriacetic acid in sewage receiving proposed new detergent formulations.

The molecular weight of sewage components is an important property for two reasons: (1) It controls the fractionation of organic components necessary to achieve ultimate isolation and identification of each, and (2) it has been claimed to be the controlling parameter in physical waste treatment processes. Molecular weight studies, both contract and in-house, are employing three techniques: membrane ultrafiltration, gel permeation chromatography, and osmometry. Comparison of the methods shows unexpected discrepancies in the apparent molecular weight values, also evidence of these fractions being complexed with metals. Early results indicate that the major part of secondary effluent organics average below 500 in molecular weight.

Of the treatment agents susceptible of elucidation by fundamental scientific research, activated carbon is the most important economically and also is most productive of useful information to guide processes, which have been largely empirical until now. The efficiency of activated carbon was found to depend on its basic characteristics, surface area, pore volume and dimensions, and surface functional groups, as predicted by theory. Other fundamental properties, not yet isolated, appear to be related to these. Apparently for the first time, meaningful information is being obtained about used and exhausted carbons, relating basic parameters to the performance of these carbons and their behavior on reactivation. In a different but related approach, thermal analysis has begun to be explored as a means of characterizing both the activated carbon and the adsorbed sewage components, as well as being used to predict reactivation behavior. These theoretical studies are aimed at establishing a sound scientific basis for carbon treatment processes now conducted with inadequate understanding.

One of the favorable characteristics of sewage, namely, that it supports well the bacterial population effecting biological treatment, is also a disadvantage, in that it makes sewage a hospitable medium for disease-producing bacteria. An intensive research program has been started to pass far beyond today's fixation on indicator organisms. Methods for assaying important pathogens in sewage -- Salmonella, Pseudomonas, Shigellae, among others -- are being developed and applied as criteria to measure the effectiveness of treatment processes in removing or destroying these organisms. Information about pathogens and indicator organisms has been assembled systematically to demonstrate the pollutional effect of primary effluent, even where BOD is not an issue.

If bacteria produce biological treatment, they can also interfere with other forms of treatment, especially physical methods, and with ultimate disposal of waste concentrates. Adverse bacterial effects have been characterized in carbon treatment (growth of pathogens) membrane processes (fouling organisms), and sludge disposal (persistence of bacteria).

The above research areas are fundamental and relatively long-term contributions to the efficiency of waste treatment processes. Since the bulk of treatment research, by other components, is immediate and necessarily empirical, this research must be guided by extensive analytical surveillance. To take advantage of the intrinsic efficiencies of specialization and centralization of advanced instruments, most of the required analyses are provided by a central analytical service laboratory, supplying about 3,000 analyses per month, distributed among some 35 methods. This support is also supplied to research contractors and grantees, including assistance in setting up and standardizing their laboratories.

To supply these services requires a constant program to select appropriate analytical methods, adapt them to labor-saving systems and instruments, develop new methods and systems for this purpose, and to shake down and calibrate these instrumental adaptations.

If the initial premise of this review is reprised -- that effective processes require adequate knowledge of the composition of starting and final materials and the way specific processes affect these compositions -- then it is apparent that the concept applies as well to full-scale treatment plants. The objective of automated control of treatment plants is accepted; such control can be accomplished only if equally automated methods of sensing composition changes can be developed. The automated instrumentation developed for volume work in the analytical laboratory is also the most promising approach to plant control instrumentation. A research effort in this direction, necessarily limited by meagre resources, has attained initial success in controlling a denitrification pilot plant by on-line analysis of nitrogen compounds and of denitrifying reagent feed. The extension of automated chemical instrumentation to full-scale treatment plant automation is a major program objective.

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SLUDGE HANDLING

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Presented by Dr. J. B. Farrell at the Dallas Symposium

Sludge has always represented the major source of problems in sewage plants and accounts for up to half of the cost of treatment (Burd, 1968, Dean, 1968). Primary sludge thickens to about 5% solids and can be digested when all goes well to remove about half of the organic matter and all danger of putrefaction. However, primary treatment is not enough for most cities and secondary treatment will be required. Waste activated sludge from secondary treatment is much more dilute and much harder to dewater than primary sludge even after digestion.

Ocean disposal of sludges is becoming less and less acceptable (Miller, 1970). A sludge, unfortunately, carries large quantities of grease and may contain pesticides and heavy metals which are objectionable in the ocean.

Incineration is a popular method of sludge disposal. It returns organic matter to the atmosphere as CO_2 and nitrogen and produces an ash that is sterile and small in volume. A good incinerator with adequate scrubbers need not produce odors or discharge anything visible to the air (Culp & Moyer 1969). High temperature incinerators operating above 1600°F such as fluid bed and slagging incinerators should be carefully checked for oxides of nitrogen which are difficult to remove in fume scrubbers. Sludge will burn without supplemental fuel if it is above 25-30% solids (Burgess, 1969). This means that about 3 tons of water can be evaporated by each ton of dry solids burned. Fuel costs are about one dollar to evaporate each additional ton of water in the sludge.

Since sludge normally contains from 20 to 100 tons of water per ton of dry solids it is necessary to dewater it as far as practical before incineration. Conventional vacuum filtration can reduce the water to 3-6 tons per ton of solids if the sludge is conditioned with iron salts and lime.

Thickening of sludge is the most economical way to remove a part of the water. Our ability to thicken sludge by sedimentation frequently exceeds the capabilities of the plumbing to remove the thickened product (Voshel, 1966). All too often we see rakes or other devices to mix the sludge with water so that it can be drawn off as a liquid in small pipes. There are also reports of sludge being diluted with water from a hose to get it out of the tank. The use of small diameter pipe with multiple right angle bends between a thickener and a sludge pump is just asking for trouble. There are real opportunities for equipment manufacturers to design improved methods for moving thickened sludge. Chemical sludges, especially lime sludges do not behave like sewage sludges and must be studied on a realistic scale before equipment to handle them can be designed (Mulbarger, et. al., 1969).

In small plants one can show that direct incineration of thickened sludge with evaporation of all of its water content would be less costly than operation of an intermediate filter or centrifuge (Culp, 1969). Some interesting work is being done on flotation (Mulbarger and Huffman, 1970). It has been well established that vacuum filter yields are proportional to sludge concentration so efficient thickening can substantially reduce filtration costs. On the other hand centrifuges may operate more efficiently with a low solids feed since sedimentation is easier in a low viscosity system (Schultz, 1968).

Polymers increase the yield or rate of filtration and improve solids capture but do not help the moisture content. Solid bowl centrifuges can also produce dry cake but solids capture may be as low as 40% unless polymers are used.

Sluige ash is an efficient low cost filter aid that is already available at the plant incinerator. Ash from multiple hearth incinerators is said to make a better filter aid than fluidized bed incinerator ash. We have found that fly ash from coal makes a less effective filter aid than sludge ash. Net yields from vacuum filters are doubled by additions of up to 200% of ash based on sludge solids. Moisture of the cake does increase slowly relative to the sludge solids so fuel costs for incineration would also rise. In Stuttgart, Germany, a vacuum filter uses sludge ash collected directly from the flue gases by a spray of wet sludge (Vater, 1969). The flue gases also heat the sludge, reducing its viscosity.

In the same Stuttgart plant there is a newly installed pressure filter using a precoat of sludge ash as a parting agent together with body feed of more ash. The press is completely automatic and produces a much drier cake than the vacuum filter so that almost no fuel is needed for incineration. In contrast to older manually operated filter presses the new presses are completely automatic even to the washing of the filter cloths. The operation is batchwise but can be fully automated. Dry sludge can be removed at the convenience of the operator once the press is full. Filter presses of this type are now available in this country.

Many other types of dewatering devices are being offered including top filters, roll presses, and tower presses. Each of these devices produces a final cake that needs supplemental fuel for burning and each requires polymer additions for efficient solids capture (Coackley, P., 1967).

Some form of stabilization of sludge before dewatering is usually required to prevent septic decomposition and odors. However, stabilization may not be necessary if the dewatered sludge is fed directly to an incinerator. The chemicals used as filter aids, especially iron salts and lime, inhibit decomposition and reduce odors while the sludge is in the plant.

Anaerobic digestion is the classical method for stabilizing sludge. When operating properly, anaerobic digesters remove heat in the form of methane, carry putrefaction to completion, and reduce the volume of solids to be filtered or dried. Unfortunately, digesters are hard to operate properly and even very large installations can be upset by trivial quantities of heavy metals or chlorinated solvents. Recent work in England has shown that as little as one drum of methyl chloroform or 1,1,1 trichloroethane, a solvent which is used for safe degreasing and cleaning, can upset 40 million gallons of digesters (Swanwick and Foulkes, 1970). Another problem with anaerobic digesters is that they solubilize a substantial part of the BOD and nutrients and return them to the plant in the supernatant soup (Barth, et al, 1966).

Anaerobic digesters work well with sludge to which iron or aluminum salts have been added to remove phosphates (Connell, 1970-71). Of course, the percent of non-volatile solids is increased. Lime in sufficient quantities to remove phosphates either in primary or tertiary treatment may produce an undigestible sludge if the pH is too high. Fortunately, limed sludge is very resistant to putrefaction at a pH above 10.

Lime sludge may be incinerated at 1800° F to recover the fraction of the lime that has combined with carbonates.

$$CaCO_3 \xrightarrow{heat} CaO + CO_2$$

Reburned lime may be better than fresh lime on a CaO basis because sludge ash in the used lime acts as a filtration aid (Mulbarger et al., 1969).

Aerobic stabilization is often used in small plants because it requires less capital than anaerobic digestion and is less likely to be upset by poisons. Power costs are definitely higher, especially if useful gas can be produced by anaerobic digestion (Ritter, 1970). Aerobic stabilization usually leads to nitrification of the ammonia. It is possible to denitrify a fully nitrified sludge by holding it for about 5 days with mild agitation without the addition of methanol or other source of BOD. Aerobic stabilization tends to hold phosphates in the sludge particles but denitrification releases some of the phosphates again to the supernatant (Randall and Koch, 1969).

Aerobic stabilization reduces volatile solids by oxidation with the liberation of heat (Kambhu and Andrews, 1969). Anaerobic digestion in contrast liberates potential heat in the form of methane which can be burned separately. The heat liberated in aerobic stabilization must be carried off as the heat of vaporization of water in the exhaust air. If insufficient air is supplied, the sludge will heat up and consume oxygen faster while at the same time the solubility of oxygen is reduced. The result is that part of the sludge will go anaerobic producing foul odors. To avoid this problem one must either supply more air or dilute the sludge. Foul odors are rarely experienced when stabilizing waste activated sludge but mixtures of activated and primary sludge will need a lot of air to carry off the heat and prevent odors. Aerobic stabilization works well with biological sludges containing iron salts or alum and has been used with Fe + CaO.

Lime has already been mentioned as a chemical stabilizing agent. Doyle (1967) has shown that raising the pH to 12 with lime will destroy pathogens in raw primary sludge although it does not completely sterilize it. Lime also inhibits odors, except ammonia, and produces a sludge that can be filtered and incinerated or can be applied to the land. If piled deeply in a land-fill this sludge would eventually putrefy but it would not contribute pathogens or nutrients to ground water. The cost will be less than digestion in most cases.

Pathogens can also be killed by very heavy chlorination or by pasteurization. Neither treatment destroys volatile organic matter and decomposition will eventually set in. Heavy chlorination could chlorinate aromatic compounds in the sludge producing toxic substances that inhibit bacterial attack and whose effect on aquatic life is unknown. Proof of safety of the effluent should be demanded before heavy chlorination is approved for sludge treatment. Pasteurization sufficient to destroy *Ascaris ova* does not improve sludge settling or filterability (Keller, 1951).

Heat treatment by pressure cooking is offered by several manufacturers. Typically, sludge is heated to about 200°C for half an hour (Brooks, 1968). If dissolved oxygen is added there will be about 15% wet oxidation of COD (Teletzke et al., 1967). The net result is a solids fraction that settles and filters very well and a supernatant soup that is rich in nutrients, BOD, and odors and is similar to digester supernatant. Wet oxidation produces a different odor from pressure cooking. The soup is a serious load on a biological treatment plant especially if dumped as a shock load; pretreatment by extended aeration or trickling filter may be necessary (Berridge and Brendish, 1967, Erickson and Knopp, 1970).

Heat treatment hydrolyzes organic matter in sludge but sludge is normally at the least efficient pH for hydrolysis. We have done experiments using SO_2 as a mild catalyst which is known to give efficient hydrolysis of proteins (Bouthilet and Dean, 1970). As expected, we get good filtration after heat treatment at temperatures below 150 °C and we get even more soluble matter in the soup. Analysis of the soup showed a high amino acid content with good representation of the nutritionally useful species. Feeding tests on rats showed that the organic molasses obtained by evaporating this soup was a good feed supplement that increased food utilization. Preliminary calculations indicate that the sale of animal feed supplement might be profitable for a large sewage treatment plant but much more work needs to be done before this becomes an acceptable process. Previous attempts to feed whole activated sludge were unsuccessful at levels above 4% of the diet (Hurwitz, 1957).

Land disposal of sludge is the oldest and least glamorous method of sludge disposal. It is also the cheapest of all methods except dumping in the ocean See Fig. 1 (Dalton, et al, 1968). Soil bacteria oxidize organic matter to CO_2 by a form of low temperature incineration which can be free from nuisances. Nutrients are recycled to crops or forests and the more refractory organic matter remains for a while in the soil as humus (Evans, 1970). The neutralizing and water holding properties of sludge can restore barren mining spoil (Knabe, 1965). Two inches of sludge has permitted grass to grow on land which was so acidic that seed carbonized as it dried out.

Sludge can be economically conveyed up to 100 miles by pipe lines from larger plants. A study at Chicago showed that pipe line and distribution costs would go down as the distance increased because land values fell off more rapidly than piping costs increased. See Fig. 2 (Bacon, 1968). Smaller plants will employ tank truck or tank car transport. Chicago currently is running sludge trains to Arcola, Illinois to empty "permanent" lagoons on prime industrial acreage. The sludge is to be distributed on mine spoil.

In addition to nutrient and fertilizer values many farms are willing to pay for sludge for its water content alone. However, the low cost of land disposal makes this process attractive even if it is necessary to lease or buy land for disposal purposes. If properly applied and if the sludge is not heavily contaminated with toxic metals, land disposal only increases the value of the soil. This is in marked contrast with dumps and permanent lagoons which remain as permanent liabilities.

The quantity of sludge which can be applied per year depends on several factors. Soil bacteria in a warm climate can oxidize up to one-inch of dry solids a month. This corresponds to 20 feet of 5% sludge each year. These oxidation rates have been obtained on a continuous basis on oily sludges in Houston, Texas for the past 8 years (Dotson, G. K. et al.,1970). Obviously, most soils could not tolerate such a heavy application of water. Actually in trials in Illinois the limiting factor in the application of digested sludge appears to be the nitrogen content (Hinesly and Sosewitz, 1970). Ammonia in fresh digested sludge is toxic to germinating seeds. Free ammonia evaporates after a few days on the field but enough nitrogen may be left to exceed the nitrogen demands of crops. Any excess is oxidized by soil bacteria and enters the ground water as nitrates.

If sludge is applied properly in thin layers and septic puddles are avoided there is little justification for pretreatment by anaerobic digestion since soil bacteria will rapidly oxidize and stabilize the organic matter (Thomas and Bendixen, 1969). However, since it may be necessary to hold sludge until it can be applied, some treatment is usually necessary to prevent putrefaction. Lime treatment and aerobic stabilization both appear to be better than anaerobic digestion for this purpose. Both control nutrients; lime holds phosphates and hastens evaporation of ammonia while aerobic stabilization with nitrification and denitrification converts organic nitrogen to nitrogen gas. The choice of a stabilization method will depend on soil and crop requirements as well as on the costs of the treatment.

Pasteurization is required in Germany and Switzerland before sludge is to be spread on pastures during the grazing season. Other countries do not require this precaution and there is no evidence that sludge properly spread on land causes disease of humans or livestock. Pasteurization costs about $6 per ton of sludge solids at the Niersverband plant near Cologne.

Salts including nitrates formed from excess ammonia may leach through the soil from heavy applications of sludge and contaminate the ground water. It is frequently possible to collect ground water in suitably located drain tiles so that it may be treated in the same way that irrigation under-drain water is treated (Tamblyn and Sword, 1969). If discharge to saline water is available, all that is necessary may be to remove nitrates. The soil is a very efficient adsorbent for all cations except sodium and lithium as well as for phosphates and organic compounds such as pesticides so the only contaminants remaining after soil filtration will be soluble salts.

Disposal of salt brines is a general problem whenever pure water is extracted from wastewater by evaporation or by any of the desalination processes such as reverse osmosis. If the ocean is nearby, it is possible to discharge salt brines without causing pollution. However, in inland areas the disposal of waste salts may contribute a major part of the cost of treatment (Rapier, 1970). Artificial or natural evaporation ponds can be used to reduce the volume of brine and the dry salts can eventually be hauled away to the ocean. Deep well injection may be acceptable in some areas but it not a general solution because of the risk of ground water contamination and even earthquakes if excessive volumes are injected (Cleary and Warner, 1970).

SUMMARY

Sludge handling will remain a costly and troublesome part of sewage treatment in spite of all the new developments. We can only hope that engineers developing new treatment processes will integrate sludge handling and brine disposal into the overall systems and not leave their residues for someone else to worry about.

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COSTS OF DISPOSAL METHODS FOR ACTIVATED SLUDGE

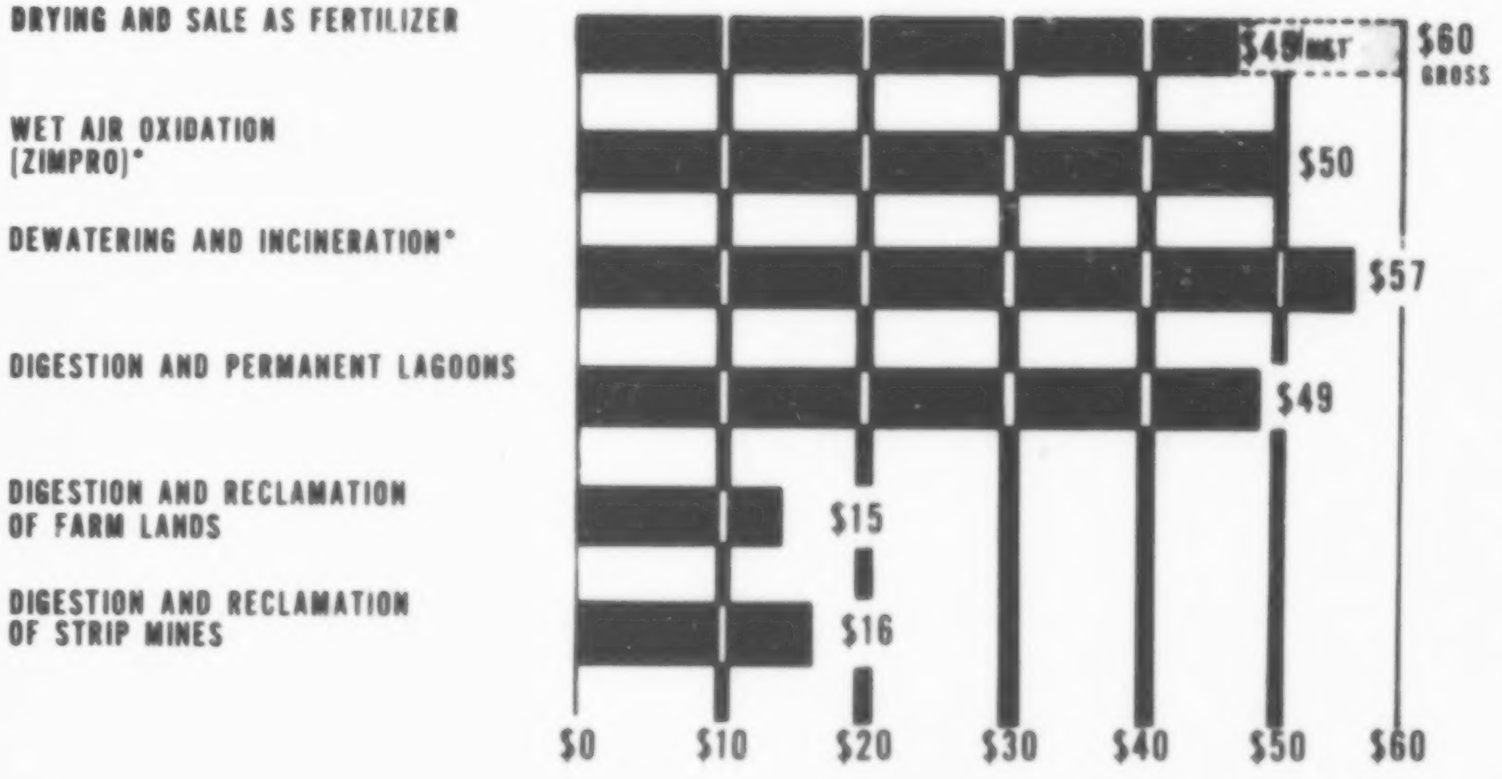

FIGURE 1

*Preceded by High Rate Digestion

LAND UTILIZATION OF SOLIDS

DIPOSAL COST vs. LAND PURCHASED PRICE

40 TON /ACRE/YEAR APPLICATION + A 50 YEAR WRITE OFF

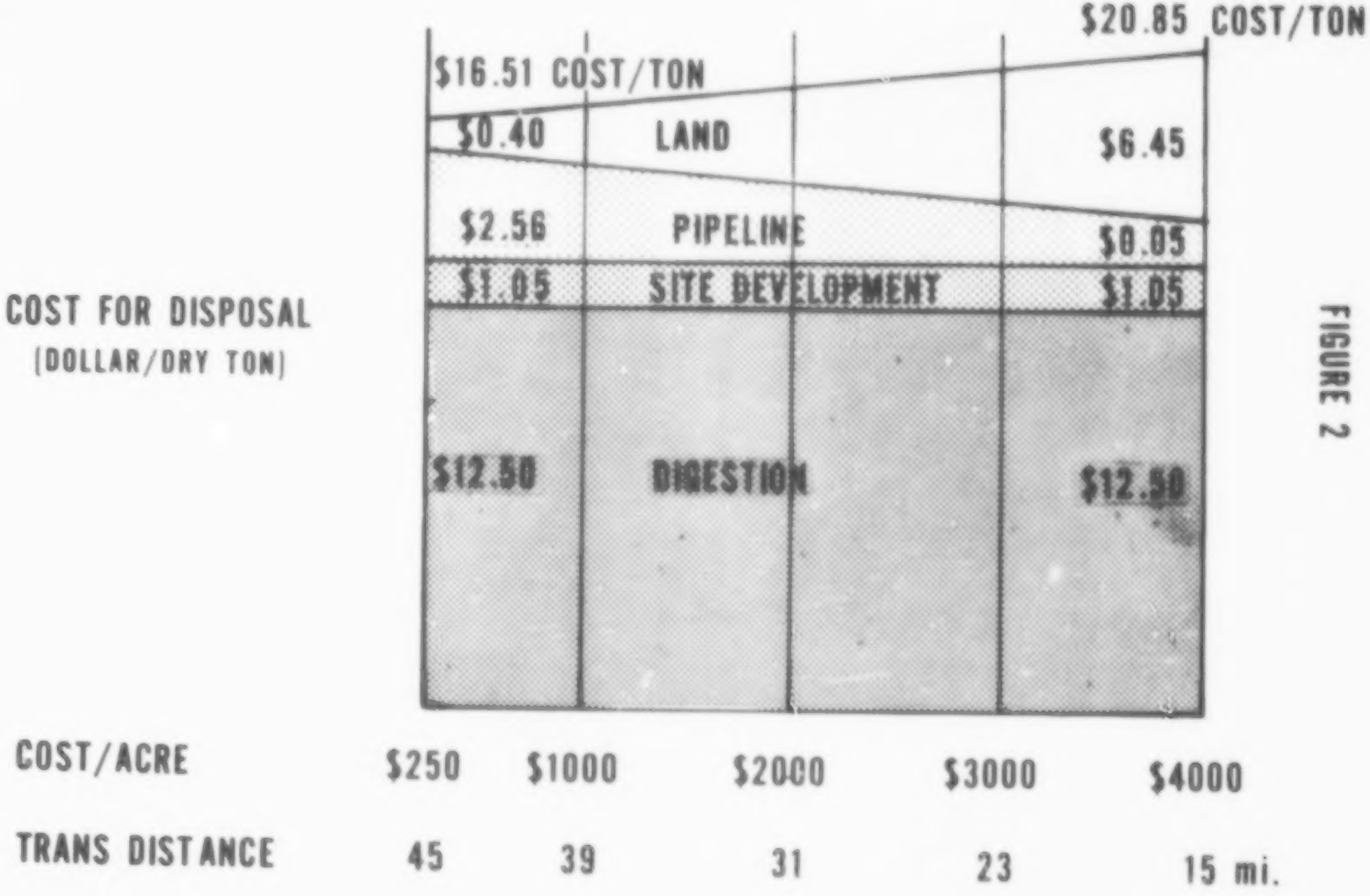

FIGURE 2

THE PORTEOUS PROCESS

BY

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CITY OF COLORADO SPRINGS
DEPARTMENT OF PUBLIC UTILITIES

November 1970

THE PORTEOUS PROCESS

In January of 1969 the City of Colorado Springs started a program of sludge conditioning with the Porteous Process. This process is based on the work done by W. K. Porteous during the early 1900's with batch process, in England.

Process operation is based on the Heat Syneresis principle. An organic sludge gel is a colloidal system incorporating within its structure large amounts of water. The structure is made up of cell walls containing cell water, and both soluble and particulate matter. Outside the cell is a gelatinous sheet composed of protein and carbonaceous material along the surface water. By heating the sludge, the cells rupture and release the entrained liquor. This phenomena is based on time and temperature, and the expulsion of the liquid from the gel is termed SYNERESIS.

After the collapse of the colloidal structure, the aqueous solution can be separated from the solids by decanting, and the solids processed by vacuum filtration, centrifuges, or presses.

The sludges now being processed in Colorado Springs are composed of 70% raw material, 10% secondary sludges, and 20% digested material. The primary raw and secondary are blended in the primary clarifiers and pumped directly to the Porteous unit; the digested sludges flow only when the pumps are not in operation from the primary clarifiers. The material now flowing from the digesters cannot truly be classed as digested since pH range, volatile acids, and alkalinity are out of the normal range of good digester operation. Instead, the material could be classed as thickened septic sludge which is only being held for heat processing. This, however, does not pose a problem, and the ultimate goal of operation is to empty the digester completely.

One of the major questions asked about the Porteous unit is, "Do sludges in the septic range, or any other configuration, cause treatment problems?" The answer so far has been, "No." It has been quite apparent in the Springs' operation that heat syneresis can be applied to any type of sludge and many other organic waste slurries. The process, being physical, is not affected by sludge type, chemical waste, or other toxic matter which so drastically affects conventional digestion systems.

Another question is, "How does the process affect filterability?" It has been determined that processed sludges where the specific resistances have been lowered to 40×10^7 sec/gm (1) will produce acceptable filterability. The Porteous produces material easily below 10×10^7 sec^2/gm, and in the Colorado Springs operation, specific resistance of 1.0×10^7 are normally found. This allows operation on vacuum filtration of 15 lbs. D.S./hr/sq.ft. of area at a moisture content of 62%. This high yield has produced two beneficial operation results. The first is the fact that two vacuum filters installed in 1965 to handle a sludge load from 20 mgd flow have been increased to 55 mgd. The second is land area for drying; under normal digester operation 24 acres had been used for beds -- with chemical treatment, this area had been reduced to 12 acres, but handling the sludges with heavy equipment was almost impossible since the material refused to dry. With the Porteous system, this area has been reduced to less than 4 acres, and piles of sludge up to 25 feet in height have been accomplished.

Another question asked is, "Does the sludge tend to reverse?" The answer to this is, "No." Since the material is heated to a point where all known pathogenic bacteria and seeds are sterilized, the material therefore, unless seeded from an outside source, remains essentially the same.

The Colorado Springs operation was built under the European flow-pattern, utilizing sludge-to-sludge heat exchange in a double-tube system. Upon startup the raw material was run through the interior tube, with the cooked material in the outside tube. After 194 hours of operation, plugging occurred, and the flow was reversed, with the raw on the outside and cooked on the inside, and again plugging occurred. Upon investigation of the sludges, it was decided that the rag content was much higher then the normal European sludges, and a water/sludge system was set up. (Figure 1.) Under this system the raw material is carried in the inside smooth tube, and water heated by the cooked sludges is recirculated in the outer tube for heat exchange.

Total process per day in this unit is approximately 108,000 gpd with an average solids range of 5%. This handles the full amount of raw material received and allows continuous removal of stored sludges in the digesters.

Probably the most maligned area of heat treatment is the production of so-called HEAT TREATMENT LIQUOR (HTL). The biological load imposed by this liquor can be treated the same as the load imposed by supernatant liquor from a digester. It is truly not an additional load but part of the function of treating wastes. However, in most installations that have been made, the design engineer would rather treat it as an imposed load, which can be summarized as follows:

$$Hs - Is = ds$$

or

$$Hs = Kds + Is$$

Where:

Hs = Strength of heat treatment liquor in BOD Mg/l

ds = Dissolved solids on ash-free basis in mgl

Is = Initial strength of supernatant in BOD_5 mgl*

K = Imposed BOD_5 per mgl of dissolved solids

*BOD_5 In the breakdown of the sludge structure a proportion of the organic matter is hydrolyzed, leading to an increase in the biological load of the sludge liquor over and above the initial supernatant strength. From a series of tests using sludges from different sources and

varying solids content, it would appear that a direct relationship can be established for the BOD_5 load imposed in relation to the milligrams-per-liter of solids taken into solution. (2)

At the Colorado Springs plant, a computer-run on eight months of operation set up a loading factor of from 8% to 10% of the influent BOD as an additional loading on the plant, not taking supernatant into consideration. (3)

In the operation of a plant, probably the worst problem with the HTL is the immediate odor while it remains hot from the heat exchanger and reactor process. This, however, drops quite rapidly as cooling takes place.

It has also been experienced that the BOD_5 is readily biodegradable and can be reduced over high-rate filters with no problems apparent. Future plan for this material is to add it to flow to the proposed activated-sludge plant, which also seems to have an excellent removal system for this type of material.

COST FACTORS

In the Colorado Springs plant, cost per ton of dry solids runs approximately \$2.10 without labor and amortization; with these factors, cost is approximately \$10.60 per ton. Trying to set a figure that would cover all installations is impossible since cost factors at each installation would be different. We do know, however, that our costs have dropped sharply from chemical sludges, which ranged from $18 to $25 per ton of dry solids, for lime and ferric alone. The other plus factors, which have not been figured in dollars, are land saving and time saving.

IN CONCLUSION, heat treatment in Colorado Springs has been an excellent asset to the operation. The mechanical difficulties that have been encountered have been far outweighed by the Process operation in producing dried sludges of a stable nature without the problems found in standard biological digesters.

JDP:rb
Att: Acknowledgements
Figure 1.

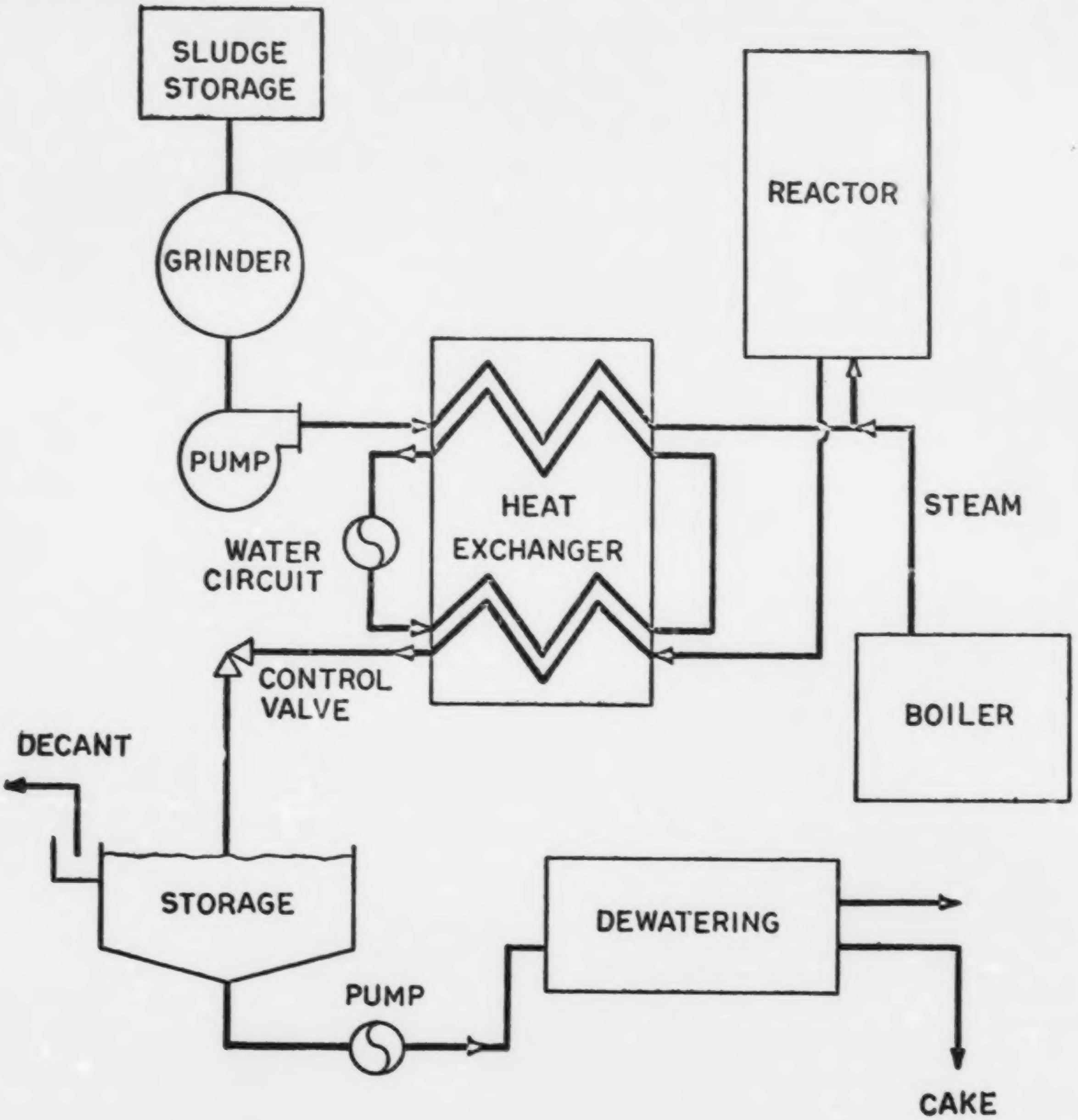
ACKNOWLEDGEMENTS

(1)	J. Swannick F. Lussignea R. Bakersville	International Conference Water Pollution Research London, September 1962
(2)	K. G. Mulhall - Associate B. D. Nicks	Discussion by the East Anglian Branch of the Institute of Water Pollution Control on Heat Treatment
(3)	Gene Suhr CH_2M Corvallis, Oregon	Predesign Report Sewerage System Improvements Colorado Springs, Colorado June 1970
Figure 1		BSP Corporation

* * *

BSP CORPORATION
AN ENVIROTECH COMPANY

FIGURE 1


SLUDGE STORAGE
GRINDER
PUMP
REACTOR
HEAT EXCHANGER
STEAM
WATER CIRCUIT
CONTROL VALVE
BOILER
DECANT
STORAGE
DEWATERING
PUMP
CAKE

SCHEMATIC DIAGRAM OF PORTEOUS PROCESS

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THE CONCEPT OF
WASTEWATER RECLAMATION

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For presentation at the 44th Annual Meeting of the Rocky Mountain Sections of AWWA – WPCA – September 1970.

INTRODUCTION

The concept of wastewater reclamation is neither new nor unique. Even in Biblical times it was known. Ecclesiastics, Chapter 1, Verse 7, states: "All the rivers run to the sea, yet the sea is not full. Unto the place from whence the rivers came, thither they return again."

Like any "new" concept, water reclamation has its friends and its foes. Both sides have many arguments supporting their positions. By way of a comparison, detractors of the Wright brothers said of man's early attempts to fly, "If God intended that man should fly, He'd have given us wings"! In answer to similar hysteria over the reuse of wastewater, we could as easily retort, "If God had meant us to use only new water, He'd either have given us more of it or created far fewer of us".

The purposes of this paper are twofold: to advance some of the many logical arguments in favor of wastewater reuse; and to cite some actual case histories of wastewater reuse operations as a means of showing what has already been accomplished.

ALTERNATIVE CONCEPTS

There is at least one alternative approach to most concepts. If we consider the reclamation concept as one possibility, its antithesis might be termed the wastewater concept. These two concepts are briefly defined below.

1. THE RECLAMATION CONCEPT

Water, once used for domestic and industrial purposes, still constitutes a natural resource that can be renovated and reused. To be justified, the reuse must be shown to be beneficial, economically feasible, and above all safe public health practice. Benefits accruable to reclamation and reuse include the identifiable economic, ecological and social impact on receiving waterways because of greatly reduced pollutional loads. These benefits are to some extent intangible since a dollar value or "price tag" for elimination of pollution is difficult to calculate. In addition, other benefits, including reduced costs for development of alternate sources of potable water supply, maximum development and use of the existing water resource, the ability to serve more people and industries and the increase of the tax base, must be considered. Failure to recognize both tangible and intangible benefits of water pollution abatement will generally result in making economic justification of water reclamation and reuse impossible

2. THE WASTEWATER CONCEPT

This approach implies continued acceptance of the premise that sewage is merely wastewater, something fit only to be discarded. This concept would logically lead to the conclusion that the pollution problem can be solved by much the same methods presently used; i. e., removal of as much of the pollutants contained in wastewater as is necessary to meet disposal requirements. Certain modifications in treatment and disposal techniques would have to be developed and implemented as pollution abatement standards become more strict, but the main idea would be to dispose of the water as a waste product, as simply as possible, consistent with the needs of the ecology of the receiving waters. Benefits accruable to this concept include the intangible aspects of pollution abatement, as in the water reclamation concept (although often to a lesser extent, depending on the purity of the effluent). Other benefits might be possibly lower costs for wastewater treatment facilities, increased stream flow for downstream users and ready public acceptance. However, the wastewater concept also has what might be termed "negative benefits," and these must be included in any logical overall comparison. These "negative benefits" are costs of developing alternate sources of potable water supply to replace effluent wasted downstream, and the possibility of placing a limit on area growth by virtue of a finite limit on available new water. Failure to recognize these and possibly other "negative benefits" may in fact constitute the major justification (and a false one) for continued waste of treated sewage effluents, at least in areas where water is a critical factor.

BENEFITS OF WASTEWATER RECLAMATION

1. POLLUTION ABATEMENT

One of the major benefits of any wastewater reclamation program is the removal of pollutants from wastewater to a degree greater than that obtainable from conventional secondary waste treatment processes. Generally, the quality standards required to make water reuse practical are such that advanced waste treatment is necessary to reduce oxygen demand; to reduce the nutrient content; to avoid undesirable algal growths; to remove final traces of suspended solids; to remove color, taste and odor; and, finally, to remove refractory materials.

Removal of pollutants causing oxygen demand is generally measured either in terms of BOD or COD removal. Items which contribute to BOD or COD in wastewaters cause a depletion of dissolved oxygen content within receiving waters. Consequently, unless the receiving water has a sufficient capacity to assimilate the oxygen demand of the incoming wastes, dissolved oxygen levels fall below desirable levels, with attendant loss of desirable aquatic fauna.

Nitrogen and phosphorus are the primary algal nutrients. Wastewaters rich in phosphorus and nitrogen stimulate the growth of algae. In clean reservoirs and streams, a delicate balance is maintained between the growth of algae and the growth of other aquatic flora and fauna. In water which receives an over-abundance of nitrogen and phosphorus, the usual result is the stimulation of greatly increased algal activity with attendant "blooms" of these minute plants. A stream or reservoir which suffers from a continuing over-abundance of nitrogen and phosphorus inevitably is on the road to degradation and eventual death as a usable body of water. This process of enrichment, because of nutrients, is termed eutrophication. Eutrophication is a naturally occurring process; eventually, the fate of most streams and lakes would be conversion first to swamps, and eventually, meadows, even without the enrichment provided by the activities of man. The rate at which eutrophication occurs is unfortunately tremendously stimulated by the activities of man, primarily because of wastewater discharges, but also to a significant degree because of the use of agricultural fertilizers on crop lands, lawns and gardens.

Suspended solids carried in wastewater effluents tend to settle in quiescent pools, in reservoirs, or receiving streams, forming sludge deposits on the bottom. These deposits are commonly known as benthol deposits and have at least two major detrimental effects. If the benthol deposits remain on the bottom, they are subject to decomposition. If this decomposition takes place in the presence of oxygen, it represents an oxygen demand. If the decomposition takes place in the absence of oxygen, malodorous conditions result. Even if suspended solids did not settle to the bottom, they represent an undesirable condition by causing increased turbidity, which decreases the aesthetic appeal of a receiving water. In addition, if present in large quantities, suspended solids have an adverse effect upon aquatic life, especially the more desirable fish species such as salmon and trout.

The presence of materials which cause color, taste or odor in receiving water probably needs little discussion since these materials are readily ascertainable to anyone who takes the trouble to use his senses.

Refractory substances, in general, may be defined as those substances which contribute to the total organic carbon content of water, but do not generally represent either a BOD or COD demand. In particular, these include synthetic detergents and many of the organic pesticides which may find their way into waters. These substances can definitely be linked to such diseases as cancer in humans and, even if not carcinogenic, may cause undesirable side effects such as foaming or frothing of receiving waterways.

2. SUPPLEMENTING THE AVAILABLE WATER RESOURCE

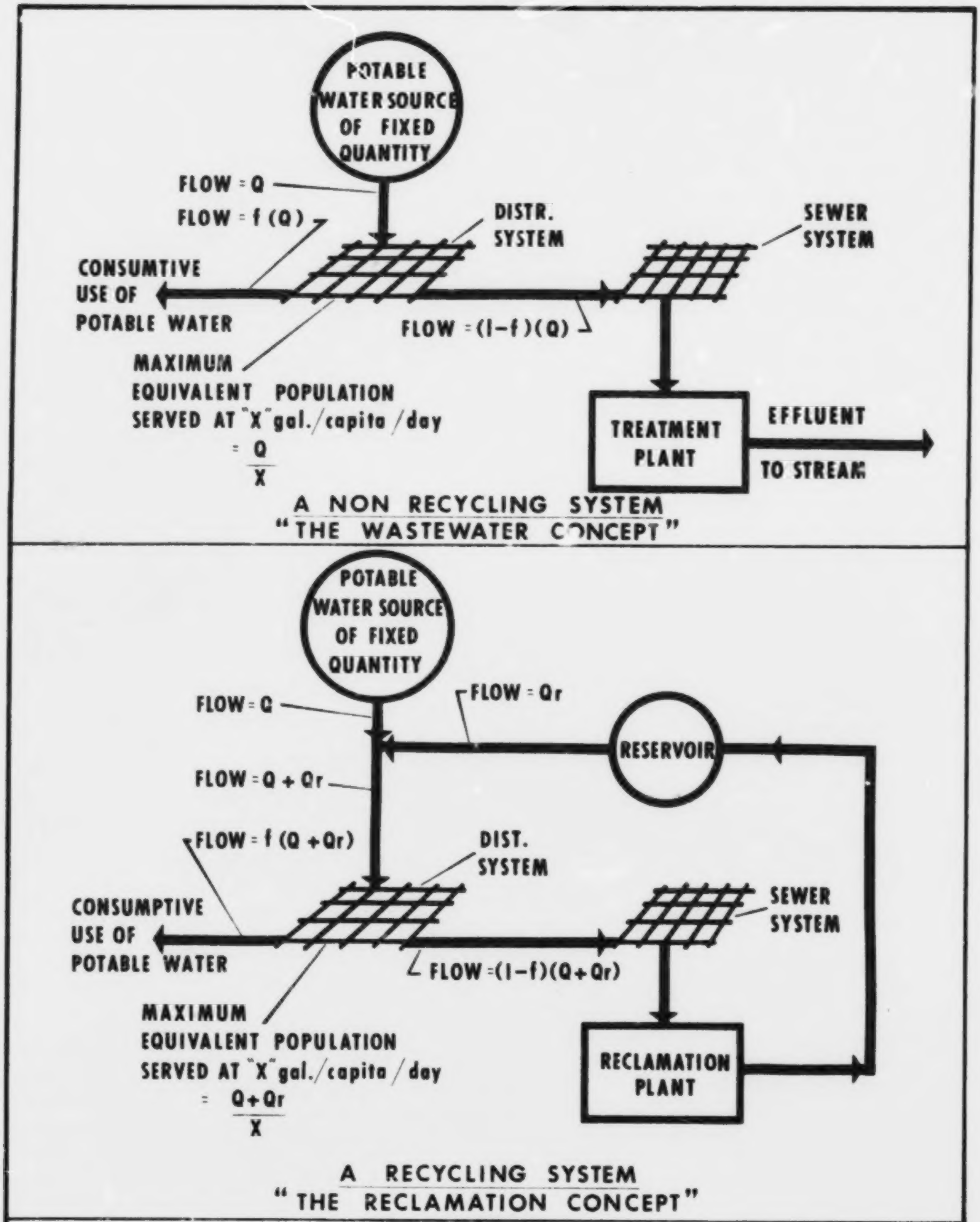
Another major benefit of water reclamation and reuse is the supplementing or enlargement of what may otherwise be a fixed quantity of water available for use to a given area. In many areas of the U. S., the quantity of water available from either surface or subsurface sources is limited; such a limit in turn imposes a limit on the total number of people and/or the total amount of industry which can be supported in a given area. Water reclamation and reuse can multiply the apparent supply by recycling reclaimed water without the necessity of developing new sources of water. Figure 1 illustrates this concept graphically.

In conjunction with water reclamation and reuse, the quality of water required for various uses often dictates the most desirable form of reclamation and reuse. For example, water for most agricultural uses and many industrial uses need not be of as high a quality as that used for human consumption. In such cases, water reclamation may take the form of irrigation of crops and use of treated effluents for industrial purposes. The net result is that the potable water supply source and system can be relieved of the burden of supplying these needs. In areas which depend largely on groundwater sources, artificial recharge of the groundwater resource either by injection or spreading of reclaimed water is often possible. This also results in augmenting the available source by essentially "banking" reclaimed water in the subsurface against the time when withdrawal of groundwater exceeds the natural recharge.

Finally, in areas where sufficient potable water is not available, highly purified reclaimed water can be recycled into the potable water system, thus increasing the overall quantity of potable water available. Such action certainly should not be implemented without considerable study and research covering all possible effects. It is not, however, an action without precedent; direct domestic recycling is now being practiced in some areas of the world and will undoubtedly become more widespread in the future. A case history involving such a direct potable water recycle is dicussed later in this paper.

3. ECONOMIC ADVANTAGES

The final major benefits of water reclamation and reuse may very well prove to be economic in nature. For example, the State of Nevada recently adopted water quality standards for the Las Vegas area which are so stringent that, if met, treated wastewater would be purer in nearly all respects than the waters of Lake Mead. However, Lake Mead, after the completion of the Southern Nevada water project, will constitute the major source of potable water for the Las Vegas area. Under the present Nevada water quality standards, desalinization of wastewater before discharge into Lake Mead would be


A NON RECYCLING SYSTEM
"THE WASTEWATER CONCEPT"

A RECYCLING SYSTEM
"THE RECLAMATION CONCEPT"

POTABLE WATER SOURCE OF FIXED QUANTITY
FLOW = Q
FLOW = f (Q)
DISTR. SYSTEM
SEWER SYSTEM
CONSUMTIVE USE OF POTABLE WATER
FLOW = (1-f)(Q)
MAXIMUM EQUIVALENT POPULATION SERVED AT "X" gal./capita/day = Q/X
TREATMENT PLANT
EFFLUENT TO STREAM
POTABLE WATER SOURCE OF FIXED QUANTITY
FLOW = Q
FLOW = Qr
RESERVOIR
FLOW = Q + Qr
FLOW = f (Q + Qr)
DIST. SYSTEM
SEWER SYSTEM
CONSUMPTIVE USE OF POTABLE WATER
FLOW = (1-f)(Q+Qr)
MAXIMUM EQUIVALENT POPULATION SERVED AT "X" gal./capita/day = (Q+Qr)/X
RECLAMATION PLANT

FIGURE 1
GRAPHIC COMPARISON OF THE WASTEWATER AND RECLAMATION CONCEPTS

required. It would therefore be far superior in many characteristics to natural Lake Mead water. Hence, unless the wastewater were to be reused it would, at least in some respects, be polluted by admixture with Lake Mead water. Thus, the very high cost of treatment would be lost for the most part, since the relative volumes are such that Lake Mead itself would be but little improved. On the other hand, reclamation and reuse plans currently under study include the use of some of the wastewater for cooling thermal power generation facilities and, in conjunction with such generation facilities, recovery and desalinization of some of the wastewater by multiple effect distillation. The desalinized reclaimed water would then be fed directly into the potable water system, actually improving the quality of the supply from the standpoint of dissolved solids content. At this point in time, the estimated net costs involved appear to be less than the cost of merely treating wastewater for disposal.

In other areas where treatment requirements may not be as restrictive, savings may still be realized by planned water reclamation. Certainly a portion of water use in nearly any sizable community may be satisfied with less than drinking water quality. The City of Colorado Springs provides such an example. For a number of years the City has maintained a nonpotable water system to supply irrigation water. This system has utilized both untreated surface water and sewage effluent that has passed through secondary treatment. Recently, the City began construction of a tertiary treatment system to augment their nonpotable water supply system. Their tertiary treatment plant will provide additional treatment for about 12 mgd of secondary effluent and produce two qualities of nonpotable water: a high grade water for cooling thermal power generation and a water of lower quality for irrigation. The system will, in addition, significantly reduce the amount of pollution entering Fountain Creek, which is the receiving stream for the effluent. Even without considering any costs for possible development of other sources of water to meet increasing demands, Colorado Springs found water reclamation to be economically justifiable. Certainly if the costs of new source development and the intangible benefits of pollution abatement were to be added, water reclamation at Colorado Springs would be even more economically favorable.

A BRIEF HISTORY OF WASTEWATER RECLAMATION

1. THE CHANUTE STORY

The City of Chanute is a relatively small community in southeastern Kansas. Its sole source of water supply is the Neosho River. Chanute maintains and operates a conventional rapid sand filtration plant to provide treatment for Neosho River water prior to introduction into the City's potable water distribution system. During the years 1953 through 1957, a record drought struck the tributary area of the Neosho; and as a result, flow in the river became progressively smaller until in early 1956 it practically ceased. Although all possible water conservation measures and limited flow augmentation procedures were pressed into use at Chanute, the water supply continued to dwindle. The situation became progressively worse until on October 14, 1956, without fanfare of any sort, City officials opened a valve which permitted mixing of sewage effluent which had received conventional secondary treatment, with water stored in the Neosho River channel behind the water treatment plant impoundment dam. During the period of water reuse, the waste treatment removed, on the average, 86 percent of the BOD and 76 percent of the COD content of the wastewater. It substantially reduced both total and ammonia nitrogen concentrations; detergent concentrations decreased an average of 25 percent. The recycling process was employed for a total of five months during the fall and winter of 1956 and 1957. It was estimated that one complete cycle through the waste treatment and back through the water treatment required about 20 days. Thus, during the total period of time during which water recycling was practiced, the same water passed through the treatment plant approximately seven times.

The treated water discharged from the water treatment plant had a pale yellow color and an unpleasant musty taste and odor. It foamed when agitated and contained undesirable quantities of minerals and inorganic substances. However, there were no known cases of water-borne disease or other adverse effects upon health resulting from the use of the recirculated water supply.

As a young sanitary engineer with the Kansas State Board of Health at the time, I vididly remember, in particular, one of the humorous anecdotes that arose from this episode. The comment was made that the water at Chanute "finally got so damn strong that we shut off the high service pumps and it pushed itself through the mains."

2. THE WINDHOEK STORY

Since the Chanute episode in 1956, the technology of wastewater treatment has indeed come a long way as represented by the latest example of reclaiming waste for domestic water supply at Windhoek in Africa.

Near the end of 1968, Windhoek, the capital of South-West Africa, became the first city in the world to practice large-scale and continuous reclamation of wastewater effluent for drinking purposes. Its wastewater reclamation plant has a design capacity of 1.2 million imperial gallons per day. Treated wastewaters constitute approximately one-third of the City's total water supply.

Prior to installation of the reclamation plant, Windhoek's main sources of water supply were a number of wells and a surface water supply from a nearby impoundment. In order to maintain the City's rate of development, augmentation of these sources of supply became necessary. The City of Windhoek is situated in a very arid region and surface water resources are scarce and expensive to develop because they involve the pumping of water over long distances. Under these circumstances, reclamation offered the cheapest solution to the problem.

The reclamation process involves the following unit process use: flocculation-flotation, detergent removal by foam fractionation, lime treatment, sterilization by chlorination, settlement of calcium carbonate sludge, sand filtration, filtration through activated carbon, and final rechlorination.

The quality of the reclaimed water easily complies with World Health Organization standards for drinking water. Excessive build-up of total dissolved solids is limited by the high natural consumptive water use in the area so that desalting is unnecessary.

No public opposition to the final scheme has become apparent.

The Windhoek plant was officially opened on January 21, 1969, by the South African Prime Minister, Mr. Vorster. The prime minister stated in his formal address that, ". . . in the future it might prove more advantageous to subsidize local authorities to reclaim their effluents than to build new reservoirs which would involve the piping of water over long distances."

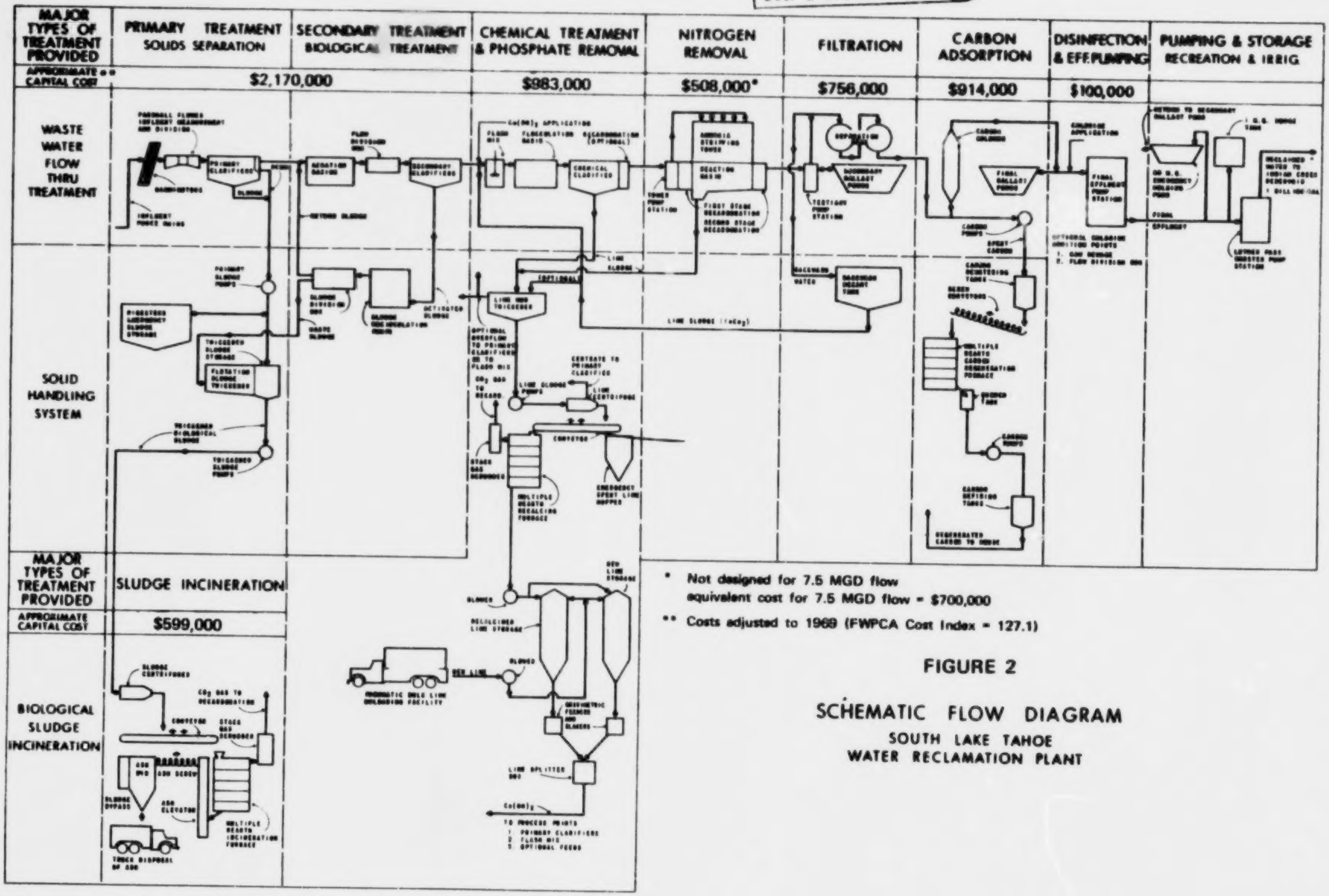
3. THE TAHOE STORY

There are only about five full-scale advanced wastewater treatment plants in the U. S. that are capable of producing the high degree of treatment required for domestic recycling of reclaimed wastewater. Perhaps the most notable of these, and certainly the first to go into full-scale operation, is the South Tahoe Public Utility District water reclamation plant at South Lake Tahoe, California.

Since March 31, 1968, the South Tahoe plant has operated without shutdown, 24 hours a day, and has continuously produced an effluent quality exceeding all the stringent requirements of regulatory agencies exercising jurisdiction in the area. At the present time, it is probably the only such facility in the U. S. which has for nearly three years produced an effluent capable of meeting drinking water standards.

Figure 2 is a simplified, schematic flow diagram of the process employed at STPUD to provide the high degree of purification and nutrient removals required. The process consists of 10 major component systems as follows:

Reproduced from best available copy.


MAJOR TYPES OF TREATMENT PROVIDED
APPROXIMATE ** CAPITAL COST
PRIMARY TREATMENT SOLIDS SEPARATION
SECONDARY TREATMENT BIOLOGICAL TREATMENT
$2,170,000
CHEMICAL TREATMENT & PHOSPHATE REMOVAL
$983,000
NITROGEN REMOVAL
$508,000*
FILTRATION
$756,000
CARBON ADSORPTION
$914,000
DISINFECTION & EFF. PUMPING
$100,000
PUMPING & STORAGE RECREATION & IRRIG
WASTE WATER FLOW THRU TREATMENT
SOLID HANDLING SYSTEM
MAJOR TYPES OF TREATMENT PROVIDED
SLUDGE INCINERATION
APPROXIMATE CAPITAL COST
$599,000
BIOLOGICAL SLUDGE INCINERATION

* Not designed for 7.5 MGD flow equivalent cost for 7.5 MGD flow = $700,000

** Costs adjusted to 1969 (FWPCA Cost Index = 127.1)

FIGURE 2

SCHEMATIC FLOW DIAGRAM
SOUTH LAKE TAHOE
WATER RECLAMATION PLANT

1. Conventional Primary Treatment
2. Completely Mixed Activated Sludge Secondary Treatment
3. Chemical Coagulation and Sorption with Lime
4. Nitrogen Removal by Air Stripping of Ammonia
5. Filtration Through Mixed Media Separation Beds
6. Granular Activated Carbon Adsorption
7. Disinfection by Chlorination
8. Coagulant Recovery by Recalcination
9. Thermal Activated Carbon Regeneration
10. Sludge Incineration

The continuously high degree of pollutant removal from the reclaimed water achieved by the Tahoe Process is shown by the data in Table 1, which gives the average overall efficiency since commencement of operations in March 1968.

TABLE 1
REMOVAL EFFICIENCY FOR THE SOUTH TAHOE
PUD WATER RECLAMATION PLANT

Parameter	Percent Removal
BOD	99.4
COD	96.4
MBAS	97.9
Phosphorus	99.1
Suspended Solids	100
Color	100
Odor	100
Turbidity	99.9
Coliform Bacteria	100
Virus	100

While the efficiency of removal of pollutants shown in Table 1 is indeed impressive, it is perhaps of greater significance to compare the average quality of reclaimed water from the Tahoe Process to various recommended drinking water standards. This comparison is made in Table 2.

TABLE 2
COMPARISON OF RECLAIMED WATER FROM THE TAHOE PROCESS AND THE RECOMMENDED AWWA GOALS FOR POTABLE WATER

Parameter		Recommended AWWA Goals for Potable Water	Reclaimed Water from STPUD Plant	Remarks
MBAS	mg/l	0.2	0.2	Meets Standard
Cl^-	mg/l[††]	250	28	Meets Standard
SO_4^-	mg/l[††]	250	36	Meets Standard
Color	units	3	0	Meets Standard
Odor	ton	None	None	Meets Standard
TDS[††]	mg/l	500	300	Meets Standard
COD[†]	mg/l	10	11	Exceeds Standard by 1 mg/l
BOD[†]	mg/l	6	1.5	Meets Standard
NO_3	mg/l as N	10	0.2	Meets Standard
S. S.	mg/l	1.0	0	Meets Standard
Turb.	mg/l	0.1	0.3	Better than most U.S. water supplies
Coliform	MNP	None	None	Meets Standard
Hardness	mg/l as $CaCO_3$	80	150	
PO_4^{-}	mg/l as P	No Standard	0.1	

[††] USPHS Drinking Water Standard – Not AWWA Goal.
[†] World Health Organization Standard – Not AWWA Goal.

Reclaimed water from the Tahoe system is not reused for domestic supply purposes. Rather, in conformance with state law, it is exported out of the Tahoe basin. However, beneficial use is made of the reclaimed water. All effluent is impounded in a new man-made reservoir located some 30 miles from the treatment plant in Alpine County, California. The reservoir has a capacity of about 3,000 acre-feet, and since its initial filling has consistently maintained a very pleasing appearance. It has been approved by local and state regulatory agencies for all water contact sports. The reservoir supports a thriving population of rainbow trout and a state grant has been awarded for construction of additional recreation facilities. Its water is sparkling clean and Secchi disc observations have been recorded as high as 20 feet. The low level of phosphorus in the water, coupled with the very low organic carbon content, appears to control algal growth adequately. During the irrigation season, a portion of the water in the reservoir is released for irrigation of forage crops by downstream ranchers in the area.

4. THE DENVER SUCCESSIVE REUSE PROGRAM

The Denver Board of Water Commissioners has long been recognized as one of the most progressive bodies in the water utility field. Their past engineering achievements in supplying high quality water to the City and County of Denver include some of the most difficult projects ever constructed. For example, the H. D. Roberts tunnel, completed in 1962, is the world's largest major underground tunnel (23.2 miles). The tunnel bore is as deep as 4,465 feet below the earth's surface and provides transmountain diversion of Blue River water to the Denver system.

With what must be termed typical foresight, the Denver Water Board has recognized that the time is approaching when even such enormous projects as transmountain diversion of west slope water will not assure totally adequate reserves of fresh water for Denver. They have therefore embarked on a research and development program to investigate and design reclamation systems to reuse wastewater. Initial plans call for a 10 mgd industrial reuse system. The Board proposes to expand the reclamation system to an ultimate capacity of 100 mgd of wastewater, with a large portion being recycled into domestic use. This project represents the largest reuse plan yet undertaken anywhere in the world and certainly is in keeping with the past achievements of the Denver Water Board.

5. OTHER RECLAMATION EXAMPLES

The foregoing examples are but a few of the many operational or projected reclamation systems. Another example of direct domestic reuse will soon be operational in South Africa with a one mgd potable water reuse plant now in experimental operation at Daspoort, Pretoria, South Africa. The Daspoort plant is very similar to the Tahoe system, using lime coagulation, ammonia stripping, and activated carbon filtration. In our own country, no listing of reclamation projects would be complete without mention of the Santee, California, project which provides reclaimed water for recreational uses. Other examples include Hyperion, California, and Long Island, New York, where reclaimed water is injected into subsurface formations to provide a salt water barrier. Finally, numerous examples of advanced wastewater treatment for pollution abatement are now under construction in the U. S. These include plants at Blue Plains (Washington, D. C.), Rocky River, Pennsylvania, and Chicago, Illinois.

PROBLEMS ASSOCIATED WITH WASTEWATER REUSE

There are still problems to overcome before wastewater reclamation, at least for domestic reuse, can become an everyday occurrence. First, the reliability of treatment processes must be improved, and along with this, the rapidity with which analysis of various pollutants can be made must be increased. Until these improvements are possible, it will probably be necessary to impound reclaimed water in reservoirs prior to release to raw water intakes at potable water treatment plants.

Bacteriological and virological testing techniques also need improvement. Today, our techniques are extremely limited. Even though extensive virological testing at both Tahoe and Windhoek have indicated no passage of viable virus through the treatment system, we are not certain that no such passage occurs, because of the difficulty of culturing any but a very few of the known viral organisims.

The progressive build-up of dissolved solids is another potential problem in water reclamation. Fortunately, this is a problem which can be solved by current technology, albeit at considerable cost, by such techniques as distillation, ion exchange, reverse osmosis and dialysis. Also, fortunately, the build-up of dissolved solids in most cases is not great due to the natural "blow-down" of dissolved solids from conventional U. S. water systems because of our prevailing rather high consumptive water use practices. The equilibrium concentration of dissolved solids that can be anticipated in any given recycling water system can be rather easily computed for any given moment if proper records are available. Figure 3 shows the general formulations necessary to solve such a problem. The technique shown was developed for computer solution with an IBM 1130 system, but it is adaptable to any computer system, or even hard calculations for that matter.

An interesting example may be obtained by using average figures for Denver (1968 data).

DATA

1968 average monthly potable water produced – 150 mgd
1968 average monthly TDS content of potable water – 135 mg/l
1968 average monthly TDS content of wastewater – 480 mg/l

ASSUMPTIONS

1. Assume 90 mgd reclaimed water recycled to domestic raw water supply.
2. Assume 50% of total flow consumptively used.
3. Assume 10% of reclaimed water consumptively used by irrigation and/or industry.

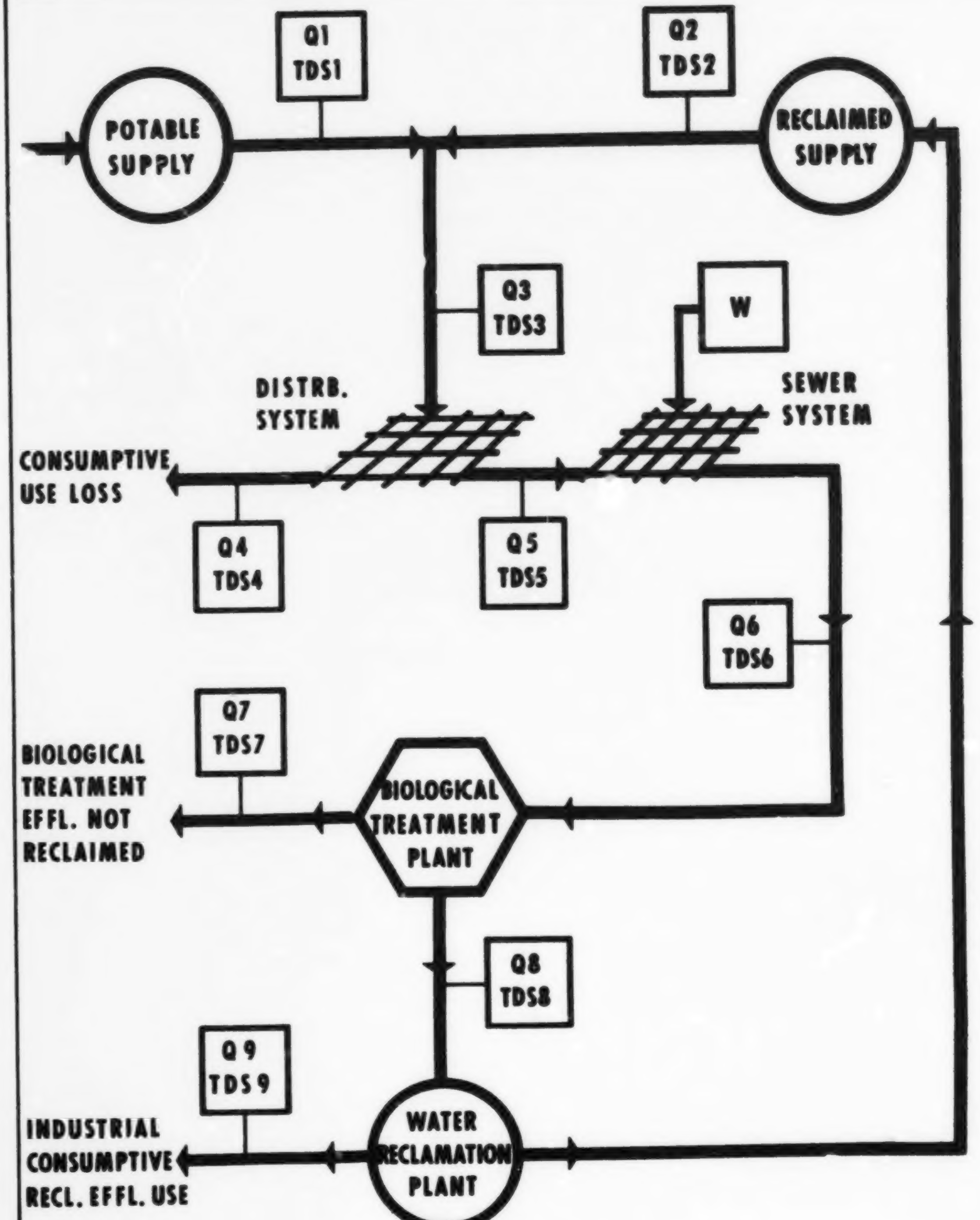

Q1
TDS1
Q2
TDS2
POTABLE SUPPLY
RECLAIMED SUPPLY
Q3
TDS3
W
DISTRB. SYSTEM
SEWER SYSTEM
CONSUMPTIVE USE LOSS
Q4
TDS4
Q5
TDS5
Q6
TDS6
Q7
TDS7
BIOLOGICAL TREATMENT EFFL. NOT RECLAIMED
BIOLOGICAL TREATMENT PLANT
Q8
TDS8
Q 9
TDS 9
INDUSTRIAL CONSUMPTIVE RECL. EFFL. USE
WATER RECLAMATION PLANT

FIGURE 3
SCHEMATIC DIAGRAM OF A RECYCLING WATER SYSTEM

NOMENCLATURE (For Figure 3)

Q1	= Flow into system from potable water supply	mgd
Q2	= Flow recycled into system from water reclamation	mgd
Q3	= Total flow to distribution system	mgd
Q4	= Consumptive water use in distribution system	mgd
Q5	= Flow to sewer system from distribution system	mgd
Q6	= Q5 = flow to biological treatment plant	mgd
Q7	= Flow from biological treatment plant not reclaimed	mgd
Q8	= Flow from biological treatment plant to reclamation	mgd
Q9	= Flow from reclamation plant to consumptive industrial use	mgd
TDS1	= Dissolved solids concentration of Q1	mg/l
TDS2	= Dissolved solids concentration of Q2	mg/l
TDS3	= Dissolved solids concentration of Q3	mg/l
TDS4	= TDS3 = Dissolved solids concentration of Q4 or Q3	mg/l
TDS5	= TDS3 = TDS4 = Dissolved solids concentration of Q3 or Q4 or Q5	mg/l
TDS6	= Dissolved solids concentration of Q6	mg/l
TDS7	= TDS6 = Dissolved solids concentration of Q6 or Q7	mg/l
TDS8	= TDS6 = TDS7 = Dissolved solids concentration of Q6 or Q7 or Q8	mg/l
TDS9	= TDS6 = TDS7 = TDS8 = Dissolved solids concentration of Q6 or Q7 or Q8 or Q9	mg/l
W	= Increase in TDS due to passage through sewer system	mg/l
X	= Decimal fraction of Q3 consumptively used	
Y	= Decimal fraction of Q6 not reclaimed	
Z	= Decimal fraction of Q8 not recycled	

EQUATIONS

$Q1 = \text{Given}$

$TDS1 = \text{Given}$

$Q2 = \text{Given}$

$Q3 = Q1 + Q2$

$$TDS2 = W + \frac{(Q1 \times TDS1 + Q2 \times TDS2)}{(Q1 + Q2)}$$

$$TDS3 = \frac{(Q1 \times TDS1 + Q2 \times TDS2)}{(Q1 + Q2)}$$

$X = \text{Given}$

$Q4 = X(Q1 + Q2)$

$TDS4 = TDS3$

$TDS5 = TDS3$

$Q5 = (1 - X)(Q1 + Q2)$

$Q6 = Q5$

$TDS6 = TDS2$

$W = \text{Given}$

$Q7 = Y(1 - X)(Q1 + Q2)$

$$Y = \frac{Q7}{(1 - X)(Q1 + Q2)}$$

$TDS7 = TDS2$

$Q8 = (1 - X)(1 - Y)(Q1 + Q2)$

$TDS8 = TDS2$

$Z = \text{Given}$

$Q9 = Z(1 - X)(1 - Y)(Q1 + Q2)$

$TDS9 = TDS2$

By using the formulations shown on Figure 3, and solving the problem for the equilibrium concentration of TDS in the blended domestic water supply, we find that such a practice would result in an increase in TDS to the consumer of 207 mg/l. Stated in other terms, drinking water at the homeowners tap would have increased from a TDS content of 135 mg/l to 342 mg/l. It would not rise above the value of 342 mg/l unless the relative volumes of 90 mgd reclaimed water to 150 mgd of "virgin" water were changed. It is doubtful if the consumer could detect such a change in TDS content, and furthermore the final TDS concentration would still be well below the USPHS allowance of 500 mg/l TDS.

Finally, we must be cognizant of public reaction to water reuse. The general public will not welcome the idea of drinking their own wastes. Experience has shown that the only way to overcome this rather natural reaction is by means of public education. The establishment of recreational reservoirs such as Indian Creek at the Tahoe project and the Santee Lakes at the Santee project have help the cause immeasurably. Such reservoirs may well be a vital key to public acceptance of wastewater reclamation; they may also be required unless or until better testing techniques are available to insure an absolute guarantee of safety for domestic reuse.

SUMMARY

In this discussion, I have attempted to show the merits of water reclamation as a concept. It may be that in some areas of the world, wastewater reuse, aside from its merits, will soon become an absolute necessity, as it already is in Africa.

We have seen what our nation's scientific and technological forces have been able to accomplish in both the nuclear and outer space areas when given unlimited financial support. If our national government supports their verbal pledges with actual appropriation of funds, we can expect that a large part of the nation's technical resources will move into the environmental control fields. Probably, in our usual manner, we will waste a lot of money and effort in our rush to develop solutions. However, there can be no doubt that the end result will be the development of many valuable new processes not known today.

The decade of the 1970's may well mark the era when America realizes that only "wastewater" fit to drink is fit to throw away. If that criterion should indeed be adopted by our society, I am certain that the technology will exist to accomplish that goal at a cost which we will be able to bear.

Finally, I would leave you with this thought: If the only water fit to throw away is fit to drink, then there is obviously no such thing as "wastewater." I believe we should discard this word from our vocabulary and along with it such terms as sewage and refuse liquids. We should come to realize that there is no such thing as new water; nearly all water on the earth's surface has been used and reused in one way or another. Perhaps we should refer to the water which carries our wastes as "transport water." Then in our treatment plants we can remove the pollutants from the water used to transport the pollutants to the plant and end up with plain water. The result might be far more acceptable to the public than trying to get them to drink "reclaimed sewage."

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